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[OFFICIAL CIRCULAR LETTER.]

Sixth Annual Convention, National Commercial Gas Association.

CIRCULAR LETTER TO MEMBERS REGARDING THE CONVENTION.—The matter in this circular, which bears the signature of Mr. W. B. Nichols, Chairman of the Entertainment Committee, gives in excellent detail the following particulars respecting the current convention of the National Commercial Gas Association, which sessions at this date are seemingly to be the Association's best. The Sixth Annual Convention of the National Commercial Gas Association will be held in Boston, at Mechanics' Building, Huntington avenue, beginning December 5, 1910.

PAPER AND ENTERTAINMENT PROGRAMME.

Monday, December 5th.—During the afternoon and evening members and their guests may register.

Tuesday, December 6th.—The business session of the Convention will begin at 10 A.M. After the business session the Exhibition will be open for private inspection by members and their guests. In the afternoon the paper programme will begin. In the evening there will be a theater party.

Wednesday, December 7th.—In the morning the paper programme will continue. In the afternoon there will be no papers, but a visit will be made to the historical points of interest in Boston. The Exhibition will be open to the public at 2 P.M. In the evening there will be a banquet for members and their guests, and at the same time a theater party for the ladies.

Thursday, December 8th.—The morning and afternoon will be devoted to papers.

Friday, December 9th.—The morning and afternoon will be devoted to papers, and the Convention will close during the afternoon. Friday, Saturday, Monday and Tuesday, December 9, 10, 12 and 13, the Exhibition will continue open for public inspection.

Hotel Accommodations.—It is important that reservations be made at once. The following is a list of the hotels, giving their accommodations, rates and plan:

		One Person in Room, with Bath.	Two Persons in Room, with Bath.	One Person in Room, without Bath.	Two Persons in Room, without Bath.
Hotel Vendome, Commonwealth Ave.	(American).	$6 00	$12.00	$5.00	$10.00 and up.
" Thorndike, Boylston Street	(European).	2.00	3.00	1.50	2.50 "
" Somerset, Commonwealth Ave.	").	2.50	4.00	2.00	3 00 "
Copley Square Hotel, Huntington Ave.	").	2.00	3.00	1.50	2.50 "
Hotel Touraine, Boylston Street	").	4.00	5.00	3.50	5.00 "
" Westminster, Copley Square	").	4.00	5.00	2.00	3.00 "
Parker House, Tremont Street	").	2.00	3.50	1.50	3.00 "
Hotel Brunswick, Boylston Street	").	2.50	3.50	1.50	2.50 "
" " " "	(American).	5.00	8.00	4 00	7 00 "
" Bellevue, 21 Beacon Street	(European).	3.50	4.50	2.00	3.50 "
" Lenox, Exeter Street	").	3.00	4.00	2.00	3.00 "
Young's Hotel, Court Street	").	2.50	4.00	1.50	2.50 "
Adams House, Washington Street	").	2.50	3.50	1.50	2.50 "
Hotel Essex, Atlantic Avenue	").	2.50	3.50	1.50	3 00 "
New American House, Hanover Street	").	2.00	3 00	1.00	2.00 "
Crawford House, 83 Court Street	").	1.50	2.00	1.00	1.50 "
Revere House, Bowdoin Square	").	1.50	2.50	1.00	1.50 "
U. S. Hotel, Beach Street	").	2.00	3.50	1.00	2.00 "
Quincy House, Brattle Street	").	2.00	3.00	1.00	2.00 "

Reservations should be made by application direct to the hotel. A map is inclosed, showing the locations of hotels in relation to the exhibition hall and the railroad terminals. Headquarters of the Association will be at Exhibition Hall.

Railroad and Boat Facilities.—Boston can be reached by any of the Trunk lines, and arrangements are being made for special rates so far as possible with the traffic associations. It is hoped to secure a round trip rate of 1½ of the regular fare. Boston may also be reached by boat from New York, Philadelphia, Baltimore, Norfolk and points in Maine and the East.

Convention Papers.—The following is a list of some of the papers to be presented during the convention:

LIBRARY OF CONGRESS NOV 14 1910 COPYRIGHT DEPOSIT.

1. "Manufacturers' Paper," Author, Wm. T. Barbour, President, Detroit (Mich.) Stove Works.

2. "Industrial Fuel," Editor, A. V. Wainwright, U. G. I. Co., Philadelphia. Sub-Editors, F. A. Tuttle, Industrial Appliance Salesman, Public Service Gas Co., Newark, N. J.; S. T. Wilson, Mgr., Improved Appliance Co., 455 Kent ave., Brooklyn, N. Y.; A. P. Ewing, New Business Mgr., Detroit City (Mich.) Gas Co.; J. F. Whaley, Travelling Industrial Expert, U. G. I. Co., Philadelphia.

3. "Illumination," Author, Norman Macbeth, Illuminating Engineer, Welsbach Co., Gloucester, N J.

4. "Service," Author, C. W. Hare, New Business Mgr., U. G. I. Co., Philadelphia.

5. "Rates," Author, H. L. Doherty, New York City.

6. "Compensation of Répresentatives," Editor, J. D. Shattuck, Gen. Mgr., Phila. Suburban Gas and Elec. Co., Chester, Pa. Sub-Editors, R. R. Young, Div. Agt., Public Service Gas Co., Newark, N. J.; C. N. Stannard, Secretary and Commercial Mgr., Denver (Col.) Gas and Elec. Co.: R. M. Searle, Vice-Pres., Rochester (N. Y.) Railway and Light Co.; A. R. Burr, Sales Agt., New Haven (Conn.) Gas Light Co.; F. J. Rutledge, Com. Agt., Phila. Gas Works.

7. "A Travelling Salesman's View of the Commercial Department." Author, H. L. Schutt, Representative, Wm. M. Crane Co., Buffalo, N. Y.

8. "Lighting and Fuel Maintenance," Editor, F. J. Rutledge, Com. Agt., Phila. Gas Works. Sub-Editors, J. Dailey, Sec'y, Pueblo (Col.) Gas and Fuel Co.; A. P. Ewing, New Business Mgr., Detroit (Mich.) City Gas Co.; H. L. Barnes, Mgr., Appliance Dept., Hartford (Conn.) City Gas Light Co.; N. W. Offut, Gen. Mgr., Mohawk Gas Co.; Schenectady, N. Y.; S. E. Linton, Agt. and Supt., U. G. I. Co., Vicksburg, Miss.

9. "Water Heaters," Joint Editors, R. C. Frampton, Pres., Pittsburgh (Pa.) Water Heater Co.; A. P. Ewing, New Business Mgr., Detroit (Mich.) City Gas Co.

10. "Relations With Customers," Editor, V. A. Henderson, New Bus. Mgr., Merchants' Power Co., Memphis, Tenn. Sub-Editors, T. F. Kennedy, Supt., Sales Dept., Denver (Col.) Gas and Elec. Co.; N. D. McPhail, Com. Agt., Western United Gas and Elec. Co., Aurora, Ills.; A. D. Dudley, Com. Agt., Syracuse (N. Y.) Lighting Co.; L. F. Blyler, New Bus. Mgr., Minneapolis (Minn.) Gas Light Co.; V. B. Day, Sec'y and Treas., Montgomery (Ala.) Light and Water Power Co.; J. J. Burns, New Bus. Mgr., Laclede Gas Light Co., St. Louis, Mo.

11. "Office and Accounting Methods," Editor, Harry Hughes, Auditor, Denver (Col.) Gas and Electric Co.; Sub-Editors, H. D. Frueauff, Gen. Manager, Pueblo (Col.) Gas and Fuel Co.; Lewis Musil, Asst. Sec'y., H. L. Doherty & Co., New York; John Lawrence, Auditor, American Light and Traction Co., New York.

12. "All-Gas Hotel Kitchens," Author, C. R. Graves, New Business Mgr., Atlantic City (N. J.) Gas Co.

In addition to these papers two other very important matters will be brought before the Association at this meeting, namely: The incorporation of the Association and the Revision of the Constitution and By-Laws.

ENTERTAINMENT COMMITTEE.

The Entertainment Committee has planned a general theater visit for members and guests (including ladies) Tuesday night, December 6th; a banquet at the Hotel Somerset, Wednesday night, December 7th, for members and guests; a theater party for ladies, Wednesday night, December 7th; a trip to historical points of interest, Wednesday afternoon, for members, guests and ladies. No other city in the country has so many prominent historical points as Boston and nearby cities, and visitors will be well repaid for trips to many interesting places.

An a la carte luncheon will be served daily during the convention at the Exhibition Hall.

In making the arrangements it is very important that the Entertainment Committee should know in advance, so far as possible, how many to provide for in the various entertainment features.

W. B. NICHOLS, Chairman, Entertainment Committee.

BRIEFLY TOLD.

ANNUAL COON HUNT OF THE MUELLER COON CLUB.—Dear JOURNAL: The "Coon Club" of the H. Mueller Manufacturing Company gave their annual "coon hunt" the evening of Saturday, November 5th. At 5:30 of that day a party of 60, comfortably seated in 10 big touring cars, left the factory in Decatur, Ills., for the hunting grounds, which are more or less eligibly located in the Sangamon river bottoms, perhaps west of Decatur some 8 miles. The air was crisp, perhaps a trifle biting, but ruddily glowing camp fires, sputtering at intervals along the trail of the hunt, served to rob the event of any real discomfort. "Bill" Hite, a famous coon hunter of pioneer days, was on hand with two trained coon dogs—just ask "Jim" T. Lynn of Detroit, to look these canines over and then think what might have happened that Saturday evening last September, on the Parry Sound dock, had "Bill's" canines been there instead of the misfits that really were there! The real hunt out-Decatur-way began about 7:30 and did not end until 11. Twenty minutes after the start the dogs captured a large 'possum, and later on ran another down; but the party was unsuccessful in bagging a coon. Although the dogs treed two, neither could be dislodged from his lofty perch. One had sought refuge in the stump of a lofty sycamore. A fire at the base dislodged the coon, but he nimbly sprang to a live part of the tree and mounted to the loftiest branch, possibly 60 feet from earth, from which coigne of vantage he viewed the crowd with very great unconcern—big "Fred." Mueller, in this connection, was heard to mutter something that eventually was transcribed to be *sang froid*. After an hour's effort he was left to his perch, much against the protest of the coon hunters, who insisted it was "poor ethics" to give up a coon after he had been treed. But "Bill" was outvoted by the hungry crowd and reluctantly called off his dogs, which had remained patiently for an hour at the tree's base, refusing to budge. The crowd, however, felt that the dogs had performed their duty, remembering with pleasure the prior deep baying as they crashed through the underbrush on the trail of the coon, the baying changing instantly to the sharp, anxious yelp that betold of the treeing of their prey. The Club had for its guests Adolph Mueller and Robert Mueller, respectively President and Secretary of the Company; Granville Meyers, Patent Attorney, Washington, D. C.; F. M. Bailey, representing *The Metal Worker*, and Thos. J. Scroggins, over whose hundreds of broad acres the hunt was held. Between 11:30 and 12 o'clock, following the hunt, a hot supper was served in the pavilion. Then came a stag dance, to the music of the Mueller Mandolin Club, a skirt dance by "Carl" Heilby, a stag quadrille, songs by the Mueller quartette, and the return home in good time and peacefully.—C. N. W.

DEATH OF DR. HENRY WURTZ.—One of the old-school chemists, a man of erudition and high scholastic attainments, a contemporary and friend of Professors Hunt, Silliman, Dana, Hitchcock and that brilliant set who, in the early '70's, electrified the chemical world with their seemingly iconoclastic outputs, and Editor of this JOURNAL in the interim of the death of Mills L. Callender and the accession of Major George Wm. Dresser (1871 to 1875), passed away when death claimed Dr. Henry Wurtz. His demise occurred on the morning of the 8th inst., at his home in Brooklyn Borough. He was in his 83d year, having been born in Easton, Pa., June 5, 1828. A history of his life will follow in due course.

PROGRESS AND EXPERIENCES IN THE FIELD OF ELECTRIC POWER TRANSMISSION AND LIGHTING-UP IN GAS WORKS.—In an article on the subjects named in the heading, prepared by Herr Gemershausen, Chief Engineer, Berlin, Germany, for the *Journal für Gasbeleuchtung*, that gentlemen goes extensively, but interestingly, into the matters noted. Having thoroughly explained and described the relative advantages and disadvantages of direct and alternating current machines, he gives his verdict in favor of the former, in so far as the use of either is permissible, in gas plants. In addition he counsels the employment of motors connected direct to steam turbines, using for fuel in the steam generators the waste material so plentiful in all works, rather than to purchase the current upon apparently advantageous terms from "outsiders," wherever that is at all possible. For all these conclusions he offers good reasons, clearly stated.—F. E.

CURRENT MENTION—

MR. THOMAS FLANDERS, since 1909 Superintendent of the La Porte (Ind.) Gas Light Company, recently resigned, presumably to take charge of the Child-Hulswit Company's properties at Columbus, Ind. His successor at La Porte is Mr. Charles Fautz.

MR. EDWIN C. WEISGERBER, Manager of the Cedar Rapids (Ia.) Gas Company, entertained his fellows in the Company at a banquet, covers for which were set in the Hotel Montrose the evening of the 31st ult. Here is the comment of one who shared its pleasures: "It was one of the very best that this famous hotel ever served, and the after-dinner programme consisted of just one speech. Ah! The beauty and joy of such a banquet as that!" The speech was surely a fitting sequel to the *menu*, for in its phrases Mr. Weisgerber dealt at length on the thought that homogenity in action was the keystone of success in any corporation, and particularly so in corporations of the type for which the Child-Hulswit Company stood.

Outdoor Lighting in England.—No. III.

[By Norton H. Humphrys.]

For some years after the introduction of the upright incandescent gas burner, it appeared that the last word had been said upon the subject. With very few exceptions the flat flame burner disappeared from our streets and thoroughfares. But in recent times a spirit of activity seems to have come into existence. The whole procedure has been under review, and many improvements have been proposed, some offering much promise, others obviously impracticable. At any rate, public lighting by means of gas has emerged from the narrow rut in which it has so long been inclosed, with the result that the gas engineer, called upon to recommend a scheme that will have to stand the test of competition, has a somewhat arduous task before him. In addition to the upright burner, there are high power lamps of the Lucas or of the Welsbach self-intensifying type, in which some endeavor is made to secure the advantages of accelerated pressure, by the adoption of a comparatively long chimney above the burner. By this means a sufficient draught can be secured to insure the combustion of 10 feet to 20 feet of gas per hour under one large size mantle, and a duty of 30-candle power or more per cubic foot of gas consumed. The Welsbach self-intensifying arrangement is largely used and with acceptable results. The light is brilliant without being harsh or glaring, and has excellent diffusive power, but clusters of two or more of these burners in one lantern, for some reason or other, are not effective in proportion. The latest favorite, however, is the inverted incandescent burner, either with or without accelerated pressure.

In large towns, where a skilled staff is available, and all the latest varieties as regards systems other than gas are in competition, the accelerated pressure is a great advantage, but in small country districts, a very good shout for gas can be made without its aid. It need not be introduced till an obvious necessity arises. I make this remark because it calls for more skill and attention than can be furnished by a small staff in which the manager alone represents the technical department. If undertaken at all, the thing must be done well, and under competent supervision.

The advance from the meter to the portable photometer, as a means of checking or ascertaining the value received, is a most important one. Making sure that at least the full specified quantity of gas is delivered and consumed is a very indirect method, and the photometer at least gives some direct information as regards value received. But illuminating value, as indicated by the photometer, is only one chapter in the story, and the results shown by that instrument do not coincide with the casual impressions of the man in the street, which really constitute the medium by which the success or otherwise is judged. The photometrical test is worse than useless, if taken at one position or angle only, because some varieties of light sources give their best results within a narrow range of not more than 20°, on the perpendicular, while others are poorly equal over the whole half circle of 180°. Whether it is left to the operator to select the best position, or a fixed angle is adopted, the result is unsatisfactory. The first condition gives power to the operator to vary the results in accordance with his own wishes, and the second may be either unduly favorable to the light or the contrary. If the maker knows the conditions under which the test will be made, he naturally directs his attention towards the best means of making a good showing on that test, rather than to the best all-round result. There is a great difference, also, between the usual method of testing a naked light contained in a dark room or chamber, and the same when *in situ* in a lantern. The form and structure of the case are so arranged as to divert either the whole or a large part of the rays thrown out in a direction above the horizontal, and reflect them downward. The inverted burner has no obstruction below the flame, and the mantle approximates to a hemi-spherical shape, so that practically the whole of the light is given off at or below the horizontal, and that in fairly equal quantity over the whole of the half-circle. This is an important advantage, which is not lessened by the fact that the shape of the inverted mantle gives a stronger section than the upright, particularly in respect to the larger sizes, and the method of suspending from a ring attached to the whole of the circumference is far superior to the loop and fork. Some makes of the inverted mantles are filled with cords of asbestos, which prevent the mantle falling when it breaks.

I do not belittle the advantage of properly conducted photometrical observations and calculations as to the maximum, average and minimum illumination per square foot of surface for comparative purposes and as a means of affording information for experts. And, perhaps, the time will come when it will be possible to set down in black and white an indisputable record of the results afforded by any particular installation of lighting. But it is useless to ignore the fact that, under present circumstances, the individual or committee who have the veto on the matter, do not attach so much importance to the results observed and tabulated by their expert, as to the judgment arrived at through their own personal impressions. They go out and look at the light from two or three different points, or ask some habitual user of the thoroughfare in question what he thinks about it. If the impressions are pleasing, little notice is taken of any intimation to the effect that theoretical conditions are not satisfied. Nor will a highly favorable report go to remove a bad general impression. So it will be desirable to give a little attention to some of the conditions that meet the public demand.

The habitual use of public thoroughfares during the hours of darkness is a modern institution. Our grandfathers locked up their houses and stayed indoors after dark, and did not venture out unless compelled to do so. With the exception of the pleasure resorts in London and other large towns the passengers were few and far between, and it was only after the introduction of better lighting and different police protection that people began to use the streets as freely after dark as during the hours of daylight. The primitive idea was that the light was a light for the feet, and that the whole requirement was to see one's way from one end of the street to the other. The roads and footwalks were in such indifferent repair that puddles or camel humps were frequent, and without the light one might get splashed with mud, or stumble over a loose stone. But now the roads are generally kept in such order that, except at crossings or refuges where one wants to see the position of the curb, this condition is not so important. When all that is required is to see the way along, beacon lights at wide intervals and at the corners of streets meet every purpose. But much more than that is at present required. There must be light enough to enable a passing pedestrian to be recognized, to read numbers on houses, names of streets, the destination of omnibuses or tram cars, to see bell pulls or knockers, pillar letter boxes, fire alarms, etc. The light must not be concentrated on the ground level, but be fairly equally diffused for at least 10 feet above. In fact, the most important is a zone of 4 to 5 feet above ground level. It is all very well to talk about the waste of lighting up the pavement and the chimney pots and of the desirability of turning the light where it is wanted, but the stopping off of the whole of the top light, as practiced in some continental cities, by using a solid-topped lantern, produces a dismal, tunnel-like effect that is depressing. In England it is usual to make the top panes of lanterns or reflectors, if such are used, of opal glass, which acts as a good reflector, and at the same time does not cut off the whole of the top light.

The ideal artificial light should be as free from any pronounced tint of color as diffused daylight, and this is a great point in favor of incandescent gas, which is in fact the only available means of meeting this condition. An ingenious apparatus known as the Tintometer is largely used in laboratories and elsewhere for the purpose of comparing, measuring and locating minute graduations of color with a great degree of exactitude, but I do not know that it has ever been taken up as a means of indicating the value of artificial light. The photometer does not indicate color except so far as the same may have any connection with illuminating value. But undoubtedly the color of the light or the absence of such has a marked effect in regard to public favor. It is a matter of public knowledge that a white light, or a reflection from a white surface, is preferable for prolonged reading, writing or drawing, than one that is decidedly blue, red or yellow. A tinted light casts its effect on all other colors, and it is a great inconvenience if common objects have two colors, one for day, the other for artificial light. So long as the flat flame, which gives a decidedly yellow tint, was used, there was a preference for elaborate fancy colored globes. Elaborate geometrical designs, or pictures of animals or landscapes, were much in favor. But the general feeling now is in favor of clear or opal globes. The "all-the-colors-of-the-rainbow" style of decoration has quite gone out of fashion. The point applies to outdoor as well as to indoor lighting.

Absolute steadiness is a necessity, as a light that is subject to frequent jumps or scintillations is not in the running, however satisfactory it may be in other respects.

The actual size or area of the light emitting surface must not be overlooked. By the use of specially shaped bulbs, or curves on the surface, this can be apparently increased, but it is the actual and not

the apparent size that tells in practice. A group of 6 or 8 loops may be made to appear like a solid, but they are not actually a solid, and do not possess the diffusive powers of a solid surface. Light for light, small concentrated areas, giving a bright spot of great intensity, are less effective in practice than a large area of less intensity. Given 2 lights, each of 500 candle power by the photometer, but one having an area of $\frac{1}{16}$ square inch, and the other an area of 8 square inches, it would follow that the intensity of the one, per equal unit of area, was 128 times that of the other. The smaller area is not equal in diffusive effect to the larger, and, according to the well-known law of contrasts, it fatigues the retina and conveys an impression of less diffusive effect than really exists. By contrast the spaces between the lights appear darker or less illuminated than is actually the case.

The best effects are produced by the use of outline illuminations, as used on the occasion of public festivals or rejoicings. The lines of light on a triumphal arch or public building, formed by means of low power units placed at a distance of 1 or 2 feet apart, give an effect that for softness, brilliancy and absence of sharp shadow, cannot be approached by widely spaced, large lights, even though the latter may show a much higher aggregate candle power. I remember seeing, in France, a street lighted by arches of pipe at frequent intervals, carried the whole width of the thoroughfare, and piped with ordinary gas burners spaced about 18 inches apart. But though the most effective, the cost is too high for everyday purposes, and practical conditions have to be considered. A single 1,000-candle light center is very much cheaper in every way—first cost, consumption of material, cost of maintenance or upkeep—than fifty 20-candle centers. Very high units are extravagant as regards light, do not lend themselves to evenness of effect and give objectionably sharp shadows. The best results in practice, having regard both to efficiency and economy, are secured by lights of higher power than can be applied for ordinary interior lighting, but still moderate rather than large, say from 100 up to 300-candle power. Such units, suitably located, spaced and environed, answer all ordinary requirements, and, except for special purposes, units over 500-candle power are not called for. It is a mistake to suppose that an excess over actual requirement is "good" lighting.

Finally the light must be reliable, not subject to more than a small percentage of failures over the year's working. One need not enlarge on the inconveniences, to say the least, that may be caused by partial or total extinction at busy times, to illustrate the fact that other advantages must be sacrified to some extent, if necessary, in favor of reliability.

The conditions that the public will appreciate, and to which the light engineer should direct his skill and energy, are reliability, absolute steadiness, freedom from any one pronounced tint of color, a fairly large area of light giving surface, and moderate power units not too widely spaced.

Brick, Concrete and Steel Holder Tanks.

[Prepared by Mr. Herbert W. Alrich, for the Fifth Annual Meeting, American Gas Institute.]

Since the inception of the gas industry there have been holder tanks constructed of many types and many materials. While tanks are still occasionally built of unusual design, the practice of the present time is represented most frequently by two types of tank and three materials. Tanks above ground and tanks below comprise the types, while brick, concrete and steel are the materials. However, these materials are not regarded as being all suitable for either type of tank. Only very unusual conditions could furnish sufficient reasons for placing a large steel tank entirely below ground, and no brick or concrete tank of large size has ever been built without a backing of earth. Hence we may state the prevailing practice to be confined either to steel tanks above ground or masonry tanks below.

The construction of the earlier gasholders required no origination in tank design. Adaptations from existing practice in cisterns were sufficient. An increase in the size of holders accompanied the extended use of gas and the construction of the tanks was successively identical with the several types of water tanks of corresponding size.

Until the '70's the tanks built in this country did not often exceed 100 feet in diameter nor 30 feet in depth. Brick, as well made as they are now, were very much cheaper. Bricklayers worked 10 hours for $2.50. At the same time wrought iron plates of the thickness necessary for tanks of suitable depth, more than 60 feet in diameter, were not being manufactured anywhere in the world. The remarkable skill with which cast iron was then applied to structural purposes testifies to the high cost of wrought iron construction at that time. Under such conditions the standard tank in this country became a brick pit, constructed either entirely below the surface of the ground or with the earth embanked against its outstanding portion. They were generally located upon a site where the ground formation was favorable.

It still sometimes happens that when planning a holder a site may be selected the geology of which is suitable for the type of tank it is desired to build. More frequently, however, we are confronted by conditions which permit little freedom, and dictate the question of the kind of tank there shall be built upon that ground. The different formations that are encountered are too numerous to classify and often too composite to describe. Among those of frequent occurrence we may mention sand, clay, rock, hardpan and mud. Given any one of these conditions or a combination of them and the advocates of brick tanks will present their claims.

It is doubtless proper to consider the advantages of any type of tank in favor of which there may be a local sentiment, but the writer is convinced that one type of tank is superior to all others and preferable for almost any conditions to be found in the United States. The brick tank seems to be regarded as the criterion of prudence and conservatism, but an examination of these claims does not disclose them as being very well founded. Brick tanks are a survival of an era of non-theoretical engineering, when the designing was done by the master mason with results which, while not dangerously bad, were not economically good. It is true that considerable mechanical reasoning was applied in the design of these tanks, but this reasoning was not developed in connection with any definite theory of the behavior of the material, and there resulted no rational mathematical determination of what, and how much, the tanks were expected to withstand. Forty years ago some formulæ were devised for computing the resistance of a masonry tank to the bursting pressure of the water. Very strangely, however, the method does not appear to have been extensively applied until engineering knowledge had advanced past the point where the assumptions necessary to the formulæ were possible to accept. All engineering structures are subject and liable to the action of forces tending to destroy them. In addition to providing for the proper resistance of those forces that a structure is intended to withstand, there must also be anticipated those other forces that may be circumstantially exerted against it.

When force is applied to a structure there follows, as a result of the elasticity of all materials, a change in the dimensions and a bending of the component parts. The consequence of these deformations is a distortion of the geometric form of the structure as a whole. If the applied force be continuously increased the deformation will correspondingly increase, until the limit of elastic behavior of the material is reached, when failure must occur. The elastic properties of all materials commonly used in engineering structures have been extensively investigated for the purpose of establishing the relation between the extension and compression of the substances and the forces whose effects these deformations are. Thus, if the forces to which a structure will be subjected and the elastic properties of the materials are known, we may, with reasonable accuracy, foretell the nature and amount of the deformations. Conversely, if we observe the nature and amount of the deformations, we may compute from them the forces acting upon the structure. Or, disregarding entirely the forces producing them, the deformations of a structure may be accepted as an index of its safety.

The distribution of the force applied to a structure among its component parts is somewhat independent of the amounts of material they severally contain. The distribution is influenced more directly by the forms of the parts and the manner in which the force is applied to them. If the amount of force applied to a structure be considerably increased, there will usually result a change in the manner of distribution. Hence, when designing, the elastic behavior of a structure must be investigated to determine the extent to which each component part participates in the work of resistance.

This investigation of the effect of force upon matter, or, in other words, of loads or pressure upon a structure is the theme of analytical mechanics. When applying this science to designing we have principally to discover the stress or amount of force exerted against each component part of the structure and the relation of this stress to the strain; that is, the shortening or elongation of the component part caused by the stress.

Since the '70's many brick tanks have been built in this country,

approximating 185 feet in diameter and 42 feet in depth. The stress conditions existing in a tank of that size will now be considered. When investigating the strength of a tank, the force we have principally to consider is the effect of the retained water. This hydrostatic pressure produces a state of stress in one of the simplest forms. It is a force acting outward, normal to the surface and uniform at all points around the entire circumference. If the wall were perfectly elastic it would become infinitely thin. Thus the nature of the stress is seen to be pure tension, though if the tank be other than truly cylindrical there will be additional stresses induced by the bending of the wall as the hydrostatic pressure tends to correct the circle.

The resistance of the tank wall is exerted against the internal bursting pressure in two ways and two only. First, the resistance of the material to the circumferential stretching; second, the adhesion of the annular bottom surface of the wall to the foundation. This adhesion to the foundation results in a restraint upon the lower part of the wall, preventing it from expanding in diameter in obedience to the internal pressure. Practical considerations always require a greater thickness at the top of the wall than that necessary to resist the tension. This excessive section materially reduces the diametrical expansion at the top. It is similar in effect to the restraint exerted upon the bottom of the wall by the foundation. An exaggerated representation of the elastic deformation of the wall is shown in Fig. 1.

Before computing the necessary wall thickness it is first necessary to determine the distance above the bottom at which the restraint of the foundation ceases to be effective. The exact determination of this distance would involve much calculation. However, with the proportions obtaining in holder tanks, this may be taken as a height lying within that on which the total *circumferential* hydrostatic pressure is equal to the resistance to shearing of the bond of the tank wall to the foundation. As this height is evidently a function of the wall thickness an assumption must be made.

It will be taken at 12 feet, making the depth from top of tank to plane of maximum ring tension 30 feet. The shear consequent upon this assumption will now be investigated. Let—

D = diameter of tank = 185 feet.
h = depth to plane of maximum tension = 30 feet.
H = depth of tank = 42 feet.
S = tension in layer of brickwork 1 inch high.
T = thickness of wall in inches.
U = permissible unit tension.
V = pressure on 1 inch of circumference 12 feet high.
w = weight of water per cubic foot = 62½ pounds.

$$\text{Then, } V = \frac{h + H}{2} \frac{w}{144} \times 12\,(H - h) = \frac{30+42}{2} \times \frac{62\ 5}{144} \times 12 \times 12 = 2{,}250 \text{ pounds.}$$

The pressure of 2,250 pounds must be resisted in shear by a sectional area of the masonry bond 1 inch along the circumference and having the thickness of the wall for its other dimension. The writer does not know of the resistance of masonry bond to this form of stress having been experimentally determined. However, judging from the ease with which brick may be loosened from the top of a wall of high class masonry, it is reasonable to consider in this connection the weight superimposed upon the lowest course of brick. In the case of a wall 42 feet high, at 120 pounds per cubic foot, the bottom brick would be under a pressure of 35 pounds per square inch. If we assume the safe resistance of the bond to shearing to equal 100 per cent. of friction there is derived for the minimum wall thickness to resist horizontal shear 5 feet 4½ inches. As we would anticipate the wall having a thickness rather greater than this the distance of 12 feet may be regarded as a conservative assumption.

Considering next the thickness necessary to resist ring tension we have:

$$S = \frac{12\,D}{2} \times h \times \frac{w}{144} = 1{,}1o0 \times 30 \times 0.434 = 14{,}450.$$

Also: $14{,}450 = T\ U$.

The solution of this last equation requires us to assign a tensional value to brickwork. All the recorded tests of which the writer has knowledge were made in compression. These, with a variation of several 100 per cent., compel us to believe that brickwork is one of the most uncertain factors in engineering.

In the designing of the brick tank 285 feet in diameter, built at Manchester, 1909, the ultimate resistance was taken as 220 pounds per square inch. This is the value given by "Trautwine" for the "Average ultimate tensile resistance of *brick*." The various compression tests have uniformly indicated that brick alone develops a higher resistance to crushing than bricks and mortar. This would seem to be necessarily true in tension, though the writer finds no record of its experimental determination. The numerous tension tests of 1:3 cement mortar indicate an average ultimate unit resistance of 175 pounds. Applying this value and 220 pounds for brick, proportionately to a vertical section of masonry, there results 208 pounds as the value of the combination.

The New York "Building Code" limits the permissible unit tension induced by bending to 30 pounds. If, however, we avoid a charge of prejudice, by solving for T with U = 100 pounds, there is derived 12 feet for the required thickness of the wall at a point 12 feet above the bottom. This would be unthinkable. Of tanks approximating the size here assumed, the writer has observed two that were 6 feet 6 inches, three 7 feet and one 8 feet thick. If we solve our equation for U taking $\frac{T}{12} = 7$ feet there results a unit tension of 172 pounds, or 83 per cent. of the probable ultimate. If the tank were slightly elliptical instead of perfectly cylindrical there would be an additional increment of tension induced by the tendency of the wall to correct its curvature when subjected to the hydrostatic pressure. This stress would be greatest at the ends of the axes, and zero at 4 points between. The question may be raised as to whether this stress is important. It is certainly safer to investigate than to guess.

In Fig. 2 there is represented the elliptical curve which the tank is assumed to have been built, superimposed upon a true circle of the same perimeter. We know from analytical mechanics that the tension in the shell of any vessel with a curved perimeter, in equilibrium, is P R, in which P = the unit pressure at any point and R = radius of curvature at that point. From analytical geometry the radius of curvature at $x = \frac{B^2}{A}$, and at $y\ \frac{A^2}{B}$.

The tension at x for equilibrium would be $P\ \frac{B^2}{A}$; the actual tension is P A. Likewise the tension at y for equilibrium would be $P\ \frac{A^2}{B}$, while the actual tension is P B. Evidently $\frac{B^2}{A} < A$ and $\frac{A^2}{B} > B$, hence equilibrium can only exist through a correction of curvature which indicates the existance of a bending moment, or internal stress in the wall must resist the bending.

As the radius of curvature of the ellipse passes through all values from $\frac{B^2}{A}$ at x, to $\frac{A^2}{B}$ at y, there is one point at which it = R. Referring again to Fig. 2, if we imagine the ellipse to change in curva ture until it coincides with the circle there is a point Z which undergoes no radial dislocation. Hence this is the point of constant curvature and no moment. Referring now to Fig. 3, we may conceive

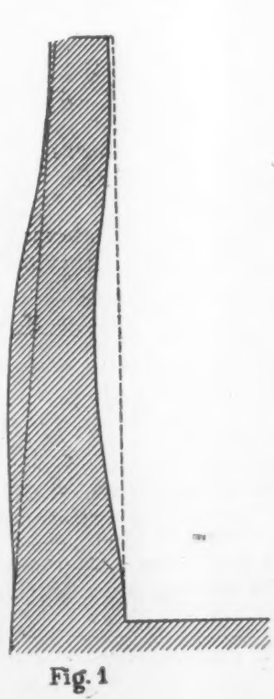

Fig. 1

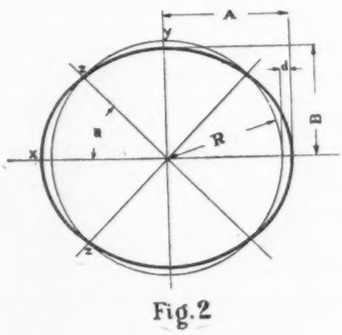

Fig. 2

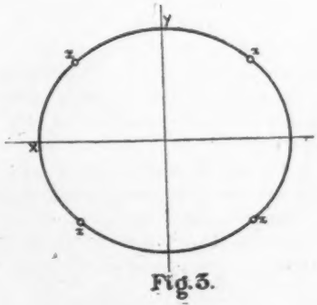

Fig. 3.

of the elliptical tank as a system of curved links, pin connected at the points Z, and acted upon by the uniform internal pressure P. Within the limits of conditions applying to this problem $\frac{A+B}{2}$ varies from R by less than any assignable difference. Also we may determine from the equations of the two curves, referred to the axes of x and y, that the angle φ does not differ materially from 45°.

Taking moments at x we have—

$$P[(\sin\phi\ R)^2 - \frac{(\sin\phi\ R)^2}{2} - \frac{(A - \sin\Phi\ R)^2}{2} - (\sin\varphi\ R)\ (A \sin\varphi\ R)] =$$

$$P[(\sin\varphi\ R)^2 - \frac{(\sin\varphi\ R)^2}{2} - \frac{(R + d - \sin\varphi\ R)^2}{2} - (\sin\varphi\ R)\ (R + d - \sin\varphi\ R)] =$$

$$P\ [0.5\ R^2 - 0.25\ R^2 - \frac{(0.086\ R + 0.586\ Rd + d^2}{2} - (0.207\ R^2 + 0.707\ Rd)] =$$

$P(-Rd - 0.5d^2)$ = moment at x.

A similar process shows the moment at $y = P\ (Rd - 0.5\ d^2)$. Hence the moment at x is negative, tending to bend the wall outward and crack it on the inside. It will be observed that the moments at x and y differ only by the value of $2\ P\ (0.5\ d^2)$ and this term is plainly negligible. Hence the moment at either point may be taken as $P\ R\ d$.

Assigning a value of 1½ inches to d the moment at a depth of 30 feet $= \frac{30 \times 62.5}{144} \times \frac{12 \times 185}{2} \times 1.5 = 21{,}700$ in pounds. With $\frac{T}{12} =$ 7 feet, then the additional increment of tension = 18.5 pounds, making a total tension of 190.5 pounds. This is 91.60 per cent. of the probable ultimate.

There have now been considered the nature and amount of the stresses imposed upon the tank wall by the internal pressure and the deformation of the masonry from the resulting strains. It is evident that for a brick tank of large size there must be provided external assistance, for any such thickness of masonry as would safely withstand the pressure unaided is economically impossible.

In the designing of brick tanks the analysis originated by M. Arson, a French engineer, still seems universally applied. This analysis first appeared in a contribution by M. Arson to the "Societe des Engenieurs Civils." It was printed in their "Transactions," about 1870. Dr. Pole translated the article and Newbigging's works gave it currency. Though the world has constantly been moving since that day, and experiment and reason have been joined to establish theory, masonry tanks are still *calculated* upon principles which, if followed to their logical conclusion, would result in a wall with twice the thickness at the top that it had at the bottom. "Calculated" we have said, not "designed," for it would appear that no tanks were ever so built. From this we may conclude that instinct has prevailed over reason and spared us the consequences of M. Arson's theory. The knowledge of the mechanics of materials in M. Arson's time was somewhat limited, hence his method disregards entirely the elastic behavior of the material. By his theory the pressure is resisted by the moments of stability, taken collectively, of a number of segmental elements to staves, of which the wall is conceived to consist, and also by the resistance of the masonry to being cracked down from the top, at each end of a diametrical line, as the tank tended to split through the center, and each half overturn.

To accept this reasoning would require us to assume that the circumferential stretching is necessarily accompanied by the wall leaning outward about the toe of the base. In other words, that there can be no outward horizontal elastic movement unaccompanied by a bodily upward lifting of the whole wall. The fallacy of this is apparent upon the inspection of any brick wall that has bulged. In fact we might deduce from the theory of long columns that the weight of the wall could assist its own destruction.

Such an analysis was made in designing the large brick tank at Manchester already referred to. The wall was imagined as being subdivided into a number of segmental elements. The summation of the moments of stability of these was assumed as co-operating with the tensional resistance of the ring in resisting the bursting pressure. An examination of the relation of stress to strain under these two conditions of loading will show that their simultaneous existence is impossible.

Fig. 4 is a cross section of the wall of three of the tanks observed by the writer. The average unit tension in a ring at the top 1 foot high $= U = \frac{\left(\frac{w}{2} \times \frac{D}{2}\right)}{12T} = \frac{\frac{62.2}{2} \times \frac{185}{2}}{12 \times 48} = 5$ pounds.

Let, E = modulus of elasticity of brickwork. d_1 equalling increment of increase of radius; that is, the outward movement of any point in the tank circumference at the top of the wall due to ring tension.

Then: $d_1 = \frac{12\pi D \times \frac{U}{E}}{2\pi} = \frac{12 \times 185 \times 5}{2E} = \frac{5{,}500}{E}$.

If an element of the wall measuring 1 foot along the circumference be now considered as a vertical cantilever, it would first deflect elastically until the adhesion of the wall to the base had been destroyed. This outward bending would not involve the wall being lifted bodily. Taking 208 pounds again as the ultimate tensile resistance of the masonry, the hydrostatic pressure, F, that the wall can withstand up to the limit of its strength, will now be computed.

Let: I = moment of inertia of cantilever.

Then: $F = \frac{3U\left(\frac{2I}{T}\right)}{12H} = \frac{3 \times 208 \times 14{,}112}{504} = 17{,}500$ pounds.

The actual total pressure, F_1 against the cantilever would be:

$$F_1 = \frac{H^2 w}{2} = \frac{42 \times 42 \times 62.5}{2} = 55{,}200 \text{ pounds.}$$

The deflection d_1, at the top of the cantilever, due to the load F, is:

$$d_1 = \frac{F\ (12H)^3}{10EI} = \frac{17{,}500 \times 128{,}024{,}064}{10 \times 592{,}704 \times E} = \frac{379{,}500}{E}.$$

The modulus of elasticity of brickwork E, is a very uncertain quantity. All attempts to determine it have produced values so unrelated as to be meaningless. Fortunately, however, the solution of our problem does not require us to assign to it any value. There have been derived two expressions for d_1; from the ring $d = \frac{5{,}550}{E}$, from the cantilever $d_1 = \frac{379{,}500}{E}$. In both of these E relates to the *same* mass of masonry, is a common denominator, and may be eliminated. Comparing the numerators it will be found that the imaginary cantilever would deflect $\frac{379{,}500}{5{,}550} = 68.4$ times as much in resisting $\frac{17{,}500}{55{,}200}$ = 31.7 per cent. of the pressure as the ring would expand in resisting all of it. From this we may deduce that the extent of this form of resistance on the part of the wall of our tank amounts to $\frac{0.317}{68.4} =$ 0.00464, or less than ½ of 1 per cent. of the pressure. In the case of the Manchester holder the proportion is very much less. This cantilever theory contains a fatal defect, in that it disregards the fact that the curve to which a cantilever deflects when loaded does not and cannot coincide with the curve to which a tank is distorted by internal pressure. Fig. 5 is an exaggerated representation of the two curves.

If we imagine a tank, built of staves rigidly fixed at the base, the hydrostatic pressure would bend them outward, opening up the joints between. If hoops were then added to close up the joints the staves would be drawn back into the position they originally occupied. The bending in them as cantilevers would then disappear and the resistance they had offered to the water pressure would vanish with it. The hoops represent the ring tension in the masonry. The only condition under which the cantilever action is approached is when the wall is made thick enough to reduce its deflection at the top to correspond to the expansion in diameter. In the case of the Manchester tank it would have been over 29 feet thick at the bottom. As tanks are built to hold water and not to practice a theory such proportions are hardly to be contemplated.

The next step in M. Arson's method is to determine the ring tension by assuming a tendency to split the wall down from the top and overrun the halves about a diametral line as shown in Fig. 6.

M. Arson's formula, still in current use, for this imaginary resistance is $U\ H^2\ T$, which he reduced from the form $\frac{H}{2}\ (2\ U\ H\ T)$, in which T = thickness of wall in feet. It could be shown that the true expression for this moment of resistance, if it actually existed, would be $\frac{2\ U\ H^2\ T}{3}$, but such computation is unnecessary in view of the fact that the resistance of the earth to the tank being depressed into it in the manner indicated by Fig. 6 would be 100 times as great. Inspection of Fig. 6 will also suggest the relation between the wall thickness at the top and bottom consequent upon M. Arson's theory. Further discussion of M. Arson's formulæ for the resistance of the

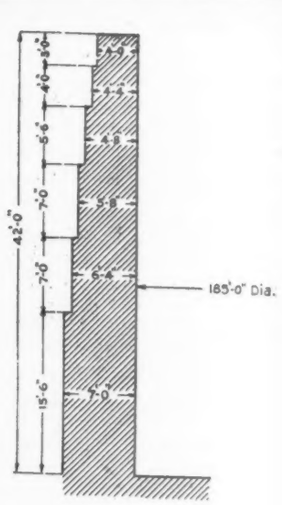

Fig. 4

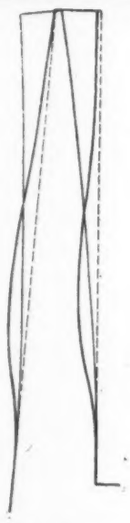

Fig. 5

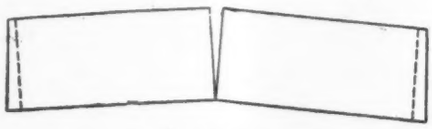

Fig. 6.

masonry seems not to be justified in view of Dr. Pole's comment in translating that it is "Only an adjunct of minor importance." However, in the calculation with which the method was illustrated 42½ per cent. of the resistance was contributed by the masonry.

"The real element of resistance is the earth backing," continued Dr. Pole, referring to Arson's formula based upon an assumption that the thrust of the earth against the tank wall has the same intensity at the surface of the ground that it has at the bottom of the well. Since engineers have known otherwise for ages it is remarkable that such an analysis ever found acceptance. It is astonishing that it is still in use. Does anyone living believe that the uppermost foot of an ordinary sand bank is capable of exerting a horizontal thrust equal to 20 times its own weight?

Arson's formula for the moment of the total earth pressure against the diametral plane = $W\ H\ (D + 2T)\ \frac{H}{2} = \frac{W\ H^2(D + 2T)}{2}$, in which W = weight of earth per cubic foot. Though we should go as far as to regard the earth as exerting full fluid pressure the moment of its thrust would then be less than $\frac{7}{8}$ of the value deduced by Arson in his numerical example.

Fig. 7 is a vertical section and plan of several segments of the Manchester tank, taken from a paper by Mr. F. H. Robinson, Engineering Assistant at the Manchester Gas Works. The paper was read before an English society and was printed in the *Journal of Gas Lighting*, November 30, 1909. When computed by Arson's method this tank had a factor of safety of but 2½. Mr. Robinson objected to Arson's method because it does not include the additional resistance offered by the adhesion of the base of the wall to the foundation, nor that which he supposed to be contributed by the buttresses. If that be the extent of Mr. Robinson's objections the remedy is a simple one: It is only necessary to extend Arson's analysis to include them. There is then derived a factor of safety of 2.9 instead of 2½.

Mr. Robinson's remedy for the low factor of safety was to investigate other stress conditions under which he deduced a factor of safety of 16. While doing this he overlooked the necessity of showing that the stress conditions assumed by M. Arson had neither a prior nor simultaneous existence. Until this is done the factor would remain 2.9. If, while investigating the strength of a simple beam which had a factor of safety of only 2.9 against failure in bending, would it add to our sense of security to discover that its resistance to shear was 16 times that imposed upon it? Right there is the defect in Mr. Robinson's analysis. He considered each panel of the wall included between two buttresses as a dam, overlooking the fundamental fact it could not so act until it had failed in tension.

An examination of Mr. Robinson's formulæ is more amusing than

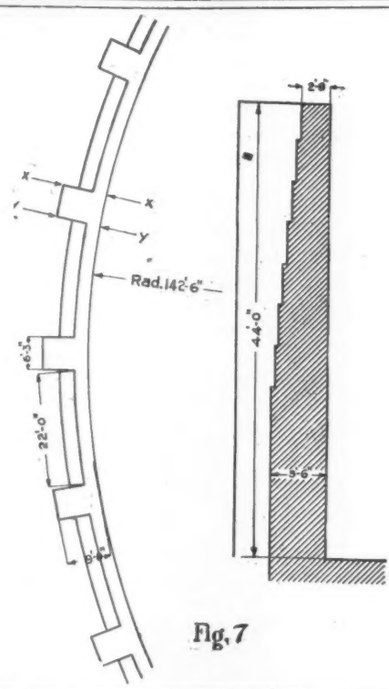

Fig. 7

profitable. They ignore not only the most elementary principles of analytical mechanics, but the laws of nature as well. These formulæ only deserve our attention because they appear to have been used in designing one of the very largest tanks in the world. As has been stated, Mr. Robinson analyzed the tank as a series of dams, each segment of the wall included between two buttresses being computed as independently stable. Taking moments about the base, Mr. Robinson assumed the following elements of resistance:

1. Moment of resistance of earth backing.
2. Moment of stability of wall due to its own weight.
3. Moment of shearing resistance through the planes $x - x$ and $y - y$.
4. Moment of resistance of the adhesion to the foundation.

The existence of these elements of resistance in the stability of a dam is fully established, but the formulæ developed by Mr. Robinson for computing their values offend ordinary mechanical instinct. Mr. Robinson repeats Arson's error of assigning the same value to the resistance of the earth backing at all depths. As such a property does not often appertain to all kinds of rock it evidently never can be possessed by earth. The acceptance of Mr. Robinson's first equation would require us to believe that the uppermost layer of earth 1 inch thick, literally the surface of the ground, would resist a force of 100 pounds per lineal foot tending to shave it off. A soil possessed of such stability would be a rare and remarkable formation. It could not be excavated by the usual methods, it would necesitate the rock drill. The assistance of a brick wall in retaining the water would be unnecessary.

It is only when Mr. Robinson's equations Nos. 1 and 3 are considered in connection with one another that his method is properly appreciated. From equation No. 1 the moment of resistance of the earth pressure against the length of 22 feet (Fig. 7) is 25,560,000 foot-pounds. Hence the moment of resistance of the earth pressure against the buttress would be $\frac{6.25}{22} \times 25{,}560{,}000 = 7{,}270{,}000$ foot pounds. The buttresses, like the wall, also derive stability from the adhesion to the foundation and from their own weight. If these elements be valued, applying Mr. Robinson's methods and unit values as disclosed by his equations Nos. 2 and 4, there results 9,270,000 foot pounds. Adding this amount to the stability contributed by the earth backing, or 7,270,000 foot-pounds, there results 16,540,000 foot-pounds as the total moment of stability of the buttresses when computed by Mr. Robinson's methods and applying his unit values. Now, this 16,540,000 foot-pounds would measure the entire extent to which the buttress could assist the adjacent wall, but Mr. Robinson's equation No. 3 contemplates an overturning moment over 16 times as great, or 276,000,000 foot-pounds being resisted by that same buttress! Thus vanishes over 80 per cent. of Mr. Robinson's factor of safety of 16. If we substitute the theories and methods of modern engineering for

those of Mr. Robinson, we find that, after the tank had failed in tension, it would have a factor of safety against overturning of about 2½.

It is interesting to observe that the investigators have confined their attention to the security of the tank against being overturned bodily. As a matter of fact, that does not concern us at all and is impossible under operating conditions. The actual task is to build a water-tight tank for no peril of collapse confronts us. Retaining walls of the usual proportions, resting upon earth, are generally found to lean outward in attaining their maximum stability. With other than very firm soil formations this leaning must occur if the wall is to develop its maximum resistance. Hence if a tank be assumed to consist of a series of retaining walls it is evident that the elements could lean outward and be stable, but the tank would not be tight. As long as the tensional resistance in the masonry bond between the elements prevents leaning there can be no retaining wall action. When this tensional resistance has been overcome the elements will be stable, but the useful career of the tank has ended.

In providing external assistance for the masonry the first resort is naturally to the surrounding earth. The particular assistance required is that which will prevent the wall from cracking. The extent to which such assistance may be derived from the backing is obviously dependent upon the character of the formation. All geological formations lie between two extremes of condition, fluid and monolithic. The vertical pressure against a horizontal plane, at a given depth below the surface of a column of fluid, is equal to the weight of the fluid above that plane. The horizontal pressure against a vertical plane of unit area at the same depth as before is likewise equal to the weight of fluid above. If the position of the vertical plane remains unchanged the pressure against it will also remain unchanged. This is the condition of passive pressure. If the plane recedes the fluid will follow and pressure against the plane will continue. This property of following a yielding resistance is active pressure. If the plane be advanced toward the fluid the pressure against it will still continue, this being the condition of bearing pressure. The vertical pressure against a horizontal plane, at a given depth below the top of a mass of rock, is equal to the weight of the rock above; but, because of the relative incompressibility and inelasticity of stone, there is neither active nor passive pressure against a vertical plane. Hence, the only pressure rock can exert against a vertical plane is the bearing pressure developed by advancing the plane against the rock.

The behavior of all earth formations lies between that of fluids and of rock. While the behavior of any particular ground cannot be foretold with much accuracy, it is somewhat consistent with the extent to which the physical condition approaches one limit or the other. If a masonry tank be surrounded by earth in a plastic condition the internal hydrostatic pressure will be largely balanced by the active external pressure, and the expansion of the tank's diameter will be reduced below that where cracking would occur. If the tank be built in an excavation in rock the backfill may be so puddled as to transmit the internal hydrostatic pressure directly to the rock. In this case also no cracking would be apprehended.

If, however, the earth surrounding the tank be somewhat firm in character, or consists of damp sand or an elastic clay, it will probably be capable of exerting little passive pressure and no active pressure. With such formations it will not avail to build up a thrust by compacting the backfill around the tank, as any such pressure in excess of that of which the earth behind the backfill is inherently capable, is artificial and cannot continue to exist.

It is doubtless a fact that active pressure exists in some of these formations before they are disturbed, but during the work of excavating the equilibrium that had existed in the mass of earth behind the face of the excavation is destroyed by the loss of those components by the earth that has been removed. If the face of the excavation then comes to a condition of repose, at a slope inclined but little from the vertical, or requiring little assistance to remain so, the forces that had composed the active pressure have become adjusted to a new condition of equilibrium in which the earth does not tend to push outward and exert a thrust against the tank wall when built.

With such formations, then, the extent to which they can assist the tank is measured by the amount of horizontally applied pressure the earth is able to withstand without compressing enough to permit the tank to expand up to the cracking point. Further, if the tank, surrounded by earth incapable of exerting active pressure, be other than truly cylindrical, when filled with water the wall will be drawn away from the earth at the ends of the greater diameter, and thereby cracking will be induced at the ends of the lesser diameter. It thus appears that some formations sufficiently firm to be conveniently excavated are very unsuitable for inclosing a brick tank. The best site for such a tank is doubtless one where the formation consists of either hardpan or stiff clay. The expense of excavating eliminates rock from consideration. While saturated ground is capable of greatly assisting the masonry, the excavation is expensive and the gelatinous condition of the soil makes probable the transmission of distant shocks.

It commonly occurs that brick tanks are cracked by the driving of piles nearby, but the writer has direct knowledge of two having been damaged by the use of explosives in removing an obstruction from a river 2 blocks away. It is now material to discover in exact terms just what assistance the tank wall shown in Fig. 4 requires of the earth. The ring tension S, at a depth of 30 feet, has been computed as 14,450 pounds. If the unit stress in the brickwork be limited to 70 pounds there will be a factor of safety *against cracking* (the only form of failure that concerns us) of about 3.

The proportion of the internal pressure thus resisted by the masonry, P_b, may be deduced from the following equation:

$$P_b = \frac{70}{\frac{S}{12\,T}} = \frac{840\,T}{S} = \frac{5,880}{14,450} = 40.7 \text{ per cent.}$$

The increment of increase in the radius of the tank d_1, due to the circumferential stretching of the wall under a unit stress of 70 pounds $= d_1 = \frac{12(D + T)\frac{U}{E}}{2} = \frac{12 \times 192 \times 80}{2\,E} = \frac{80,640}{E}$.

As has been stated, investigation has failed to determine any satisfactory value for E, the modulus of elasticity of brickwork. However, it is logical to assume this modulus as not varying materially from that of the cement mortar, which modulus has been found to lie between the limits of 500,000 pounds and 1,500,000 pounds. If we assume the average of these values and solve the equation $d_1 = \frac{80,640}{E}$ there results 0.0806 inch as the amount the brick wall at a depth of 30 feet advances outward in performing its share of the work.

The outward pressure of the water on 1 square foot of wall at a depth of 30 feet is 30 by 62.5 = 1,875 pounds, of which 100-40.7, or 59.3 per cent., must be resisted by the earth. Hence, we may conclude that, in the absence of either active or passive pressures, the tank will be secure against cracking if the earth backing be capable of developing a resistance of 0.593 by 1,875 = 1,110 pounds per square foot while compressing not more than 5/64 of an inch. It is difficult to believe that such a condition has been frequently realized. The practice of reinforcing brick tanks with steel bands would indicate that the builders of tanks have not found the earth backing a very dependable assistance.

Of the six masonry tanks approximating 185 feet in diameter observed by the writer, three are reinforced with steel bands. These bands are all ⅝ inch thick and about 5 feet wide. In apparent compliance with Arson's theory these bands were placed just below the second offset from the top of the wall. It is a fact of much significance that, though these tanks rest on bed rock and are surrounded by very firm ground, one of them has cracked below the band.

In designing these tanks it was doubtless assumed that there would be imposed upon the band a tension approaching the maximum safe stress on the metal. As a matter of fact such a result could only be attained by cracking the masonry. Assuming the unit tension in the brickwork to be limited to 70 pounds and the modulus of elasticity to be 1,000,000 pounds, the maximum stress possible in the steel may be deduced from the relation of the modulus of elasticity of brickwork to that of steel. The accepted value of E for steel is 29,000,000 pounds. Applying these values the unit tension developed in the steel under the conditions assumed would be $\frac{29,000,000}{1,000,000} \times 70 = 2,030$ pounds. With the brick stressed up to the breaking point the unit tension in the steel would be only about 6,000 pounds.

With average unit stresses of 2,030 pounds in the steel and 70 pounds in the brickwork, the band will co-operate with a ring of masonry, 7 feet 1 inch high, in resisting the hydrostatic pressure included between the planes coinciding with the top and bottom of the masonry ring, as shown by Fig. 8, the highly-stressed lower courses of brickwork receiving no assistance from the band. The total tension in the ring and band developed by the internal pressure is 400,000 pounds. With a joint efficiency of 80 per cent. the band would be capable of resisting the entire tension while under a unit stress of

only 13,500 pounds. Though we assume any other possible stress for the masonry, and any other probable elastic relation between it and the steel, or any other vertical position for the band, similar results will follow, and we shall find that whatever tensile resistance can be developed by the combination of brick and steel can be secured from the steel alone. Hence, if the brick tank here discussed were reinforced with steel bands properly proportioned to keep the unit stress in the masonry below 100 pounds, it would require as much metal as a steel tank.

An established feature of modern engineering is the preference, in planning large structures, for those designs and types in which the expectation of security is based upon faith rather than hope. This may also be stated by saying that engineers avoid those designs in which the stress conditions resulting from the system of loading governing the design are either materially uncertain or not susceptible of determination. The most frequent occasion in gas engineering for applying this criterion is the designing of holders. If the criterion should be applied even in moderation to tank design but few brick tanks would ever be built.

The writer has observed clay formations of such stability that they could not be excavated by ordinary methods. These deposits were being quarried for use in the ceramic arts. If such a formation should be found upon a site selected for a holder a strong and inexpensive tank could be constructed by merely lining a circular excavation with a thin brick wall constructed with suitable piers for supporting the guide frame. The cost of the tank would be largely defrayed by the sale of the clay. Such were the circumstances at Manchester, the clay having been made into a high-class brick at the site. In view of the fact it is strange that the designers exerted themselves to devise a fictitious resistance for a wall palpably incapable of taking even a fifth part of the internal pressure.

The writer has knowledge of an engineering work where the test borings disclosed a subsoil of hardpan possessing such stability that the contractors bidding on the work were advised to estimate the excavation as being in rock. If such a formation as this should be encountered in locating a holder a tank economical and strong could be built as before, by merely lining the excavation and providing piers for supporting the guide frame. For a tank to be so constructed it would be necessary that the formation of clay or hardpan should extend considerably beyond the tank area and evidently any disturbance of the adjacent ground would be a serious matter.

The idea seems to be prevalent that the first cost of brick tanks is in general less than that of steel tanks. It is the writer's belief that a brick tank can cost less than a steel tank only when the work begins with favorable conditions and is accompanied by them as it proceeds. If the conditions require much sheet piling and pumping the cost of a steel tank is certain to be exceeded. While plain concrete tanks were built more than 30 years ago, reinforced concrete tanks have only appeared in the last decade. These reinforced tanks represent three different types of construction:

1. Tanks with plain annular walls, the concrete being assumed to participate in the work of resistance.
2. Tanks with relatively thin walls, provided with outstanding piers for supporting the guide frame, the steel reinforcement being assumed to take all the tension.
3. Tanks similar to No. 2, but with relatively thinner walls, belted with hoops of reinforced concrete, the steel being assumed to take all the tension, as before.

There are two other types for which the writer has seen designs, but has no information as to their having been built. One of these types is designed with a wall similar to the familiar retaining wall with counterforts. In the other type the wall consisted of a ring of arches in a vertical plane, sprung between buttresses that were assumed to receive the outward thrust of the arches and hold them in together against the hydrostatic pressure. In neither case was the wall as designed capable of offering much resistance to tension. Hence, unless a tank built from either design rested upon solid rock or was surrounded by a very firm earth formation, it would certainly crack; for absence of circumferential tension would require the counterforts or buttresses to be absolutely unyielding. If they were not thus rigid their outward movement would be necessarily accompanied by an increase in circumference, and any such increase means tension. If the wall were surrounded by a formation of firm clay or hardpan the backing would probably be capable of withstanding the entire outward pressure, with less compression than would cause the concrete to crack, but the buttresses or counterforts would be useless and contribute nothing whatever to the strength of the tank.

At the time the two 300-foot tanks at Astoria were being designed Mr. Bradley and Mr. Bruce made an exhaustive investigation into the question of concrete tank construction. The writer was employed upon the work as draughtsman and computer. There being practically no precedence to consider, various types of construction were devised, then analyzed and compared. The conclusion was finally reached that the concrete tank most easily and quickly constructed, at the least cost, and in service the most reliable was the type already described as No. 1. The experience accrued during the construction of the two tanks at Astoria, and the tank 189 feet in diameter built for the Central Union Gas Company fully justifies the conclusion that this type No. 1 is preferable to any other design of concrete tank.

The disadvantages common to both types 2 and 3 are too obvious to require much discussion. If all the tension is to be taken by the reinforcement then evidently the quantity of steel in the hoops and vertical bars will exceed that sufficient for the shell of an ordinary steel tank. To whatever extent reinforcement in place may cost less than the equivalent plate work, the economy is more than offset by the cost of the concrete. The writer believes it has been fully demonstrated that, while the cost of any type of concrete tank must exceed the cost of an equivalent steel tank, the cost of types 2 and 3 is prohibitive.

The structural objections to types 2 and 3 are: First, the care required to properly locate the reinforcement and secure it against displacement; second, the great difficulty of reducing the concrete to a compact mass when its flow is obstructed by the mesh of the reinforcement. Yet, whatever strength such a tank can possess is measured directly by the extent to which both of these requirements have been accomplished. The design of a tank of type No. 3, which was built in 1908, called for a circumferential reinforcement of $\frac{3}{4}$ in. bars, space $2\frac{1}{2}$ ins. center-to-center, and $\frac{1}{2}$ in. vertical bars spaced $11\frac{1}{2}$ ins. apart. It is alleged that the work was executed in accordance with the plans. Hence, unless the builders resorted to a process resembling dentistry, we are expected to believe that the concrete when tamped flowed through a space $1\frac{1}{4}$ ins. wide by $11\frac{1}{2}$ ins. long and became thoroughly compacted around the bars. Fortunately this tank is surrounded by saturated ground.

Within the last few years the Board of Public Works, of New South Wales, built four water tanks differing in design from type No. 2 only in the omission of piers. In 1909 one of these reservoirs totally collapsed. Though designed for a head of 40 feet, it contained at the moment of failure but 32 feet of water. The engineers who investigated the collapse reported the two principal causes of failure as being: First, the concrete had not been thoroughly bedded around the bars, under which condition the tensile resistance of the steel could not be developed; second, the filling of the tank began "only 98 days after its completion."

The engineers did not state what in their opinion should have been the age of the tank before filling, but it is possibly indicated by a series of tests they had made in which specimens 98 days of age were compared with others six months old. If a concrete tank requires any such period of seasoning then the significance of that necessity is best realized by considering the fact that large holders with steel tanks can be erected complete in six months.

The analysis of stresses for a concrete tank of type No. 1 is substantially the same as that which has been applied to brick tanks. In this case also the secondary stress resulting from ellipticity may be a serious matter. With great care and much supervision during construction any considerable ellipticity may doubtless be avoided. However, the writer believes that there is sufficient probability of a tank being other than truly circular to justify an assumption that d, the increment of ellipticity, has a value of $\frac{1}{4}$ inch per 100 feet of tank diameter.

In the discussion of brick tanks there was considered the restraint exerted by the foundation upon the lower part of the wall. In a concrete tank of type No. 1 this restraint may be safely assumed to be effective for a distance above the tank bottom equal to 1.75 times the thickness of the wall. Hence, the maximum wall thickness and the depth to plane of maximum tension are reciprocally dependent. A preliminary computation of the wall thickness may be made, assuming the plane of maximum tension to be 0.8 of the depth below the top of the tank.

Evidently the consequence of this restraint upon the wall is a shearing stress in the plane of attachment to the foundation. The responsibility of resisting this stress should be imposed upon vertical bars imbedded in the foundation and carried up 5 feet into the wall.

The proper unit-tress for concrete holder tanks is still an open question. There has as yet been neither sufficient experience nor investigation to justify any very positive conclusions. In a tank of type No. 1 the stress in the steel is dependent upon the elastic relation of the steel to the concrete. The modulus of elasticity of concrete is an indeterminate quantity. Numberless tests have been made giving values of from 1,500,000 pounds to 5,000,000. The building department of New York City assumes the ratio of the modulus of elasticity of concrete to that of steel as being 12, but 15 is the ratio most frequently assumed by engineers when not restricted by ordinances.

The unit-stress that may be safely assumed for the concrete is somewhat dependent upon the character of the surrounding earth. For instance, if the tank be located in saturated ground we may with considerable accuracy compute the minimum value of the active pressure that will act inward against the wall. Hence, if the unit tension in the concrete were assumed at 90 pounds we might expect with reasonable certainty that that stress would never be exceeded. If, however, the tank be surrounded by damp sand it would be hazardous to assume such a stress as 90 pounds, unless the sand be regarded as incapable of either active or passive pressure. With the earth pressure assumed in designing the Astoria tanks the maximum tension in the concrete would be 83 pounds, but from the character of the formation as disclosed by the excavation the writer would conclude that it does not exceed 25 pounds.

In the case of tanks of type No. 1, under 175 feet in diameter, the wall thickness will generally be determined by the width at the top necessary to accommodate the base of the guide frame standards. This thickness, if continued down to the bottom, may sometimes be in excess of that necessary to resist the maximum tension. Hence it may be suggested as an economy that the wall thickness be reduced and piers added to the outside of the wall for supporting the guide frame. It can be shown, however, that such a modification may result in increasing the cost of the work. Tanks of type No. 1 are usually constructed by moulding the concrete between two concentric rings each consisting of segmental forms about 6 feet high. As the work proceeds these forms are slid upward in vertical guides. It is apparent that outstanding piers will interfere seriously with the continuity and facility of this process and offset the economy in concrete. Also, from a frequent experience with buttressed retaining walls, we may conclude that the corners formed between the wall and piers favor the development of cracks. The design of type No. 1 is based upon a theory supported by experience that it is cheaper to put in more concrete and avoid complicated form work and tangles of reinforcement.

In proportioning the reinforcement for type No. 1 it is not advisable to exceed 2½ per cent. in any unit height of the vertical section. If this amount be materially exceeded the bars will be spaced too closely to permit the best results in placing the concrete, and the additional steel not being economically employed will increase the cost very perceptibly. The concrete in the upper half of the wall will rarely require the assistance of reinforcement in resisting the ring tension, but ½ of 1 per cent. of steel in that part of the wall will do much to prevent shrinkage or temperature cracks.

Except in the unusual case of a tank having earth formations radically different in behavior, acting against opposite arcs of its circumference, any computation of the tank's resistance to externally applied pressure is unnecessary. Though we should assume an external fluid pressure from a saturated earth weighing 110 pounds per cubic foot, the maximum unit compression in the concrete would not exceed 600 pounds.

Only very small tanks of types Nos. 2 and 3 have as yet been built. These tanks have most of their depth above the surface of the ground. As has been stated, these types of construction are expensive, hence for economy it has been the practice to employ the steel at stresses as high as 25,000 pounds. When the stress in the steel reaches 5,000 pounds every vestige of the tensile resistance of the concrete has been destroyed. It is not this fact in itself that concerns us, however, but the attendant condition of the concrete. It has been established by experiment that when concrete is thus strained there is developed a countless number of cracks which, though invisible, are not less real. In this condition the concrete is not impervious to water and it has been found that a mere seepage through the wall will eventually become a flow. Furthermore the alternate freezing or thawing of the water in the cracks results in a gradual splitting of the concrete.

In 1907 the Board of Water Supply, New York City, made extensive tests with a reinforced concrete cylinder 11 feet in diameter and 210 feet long. The walls of the cylinder were 8 inches thick and contained 5 per cent. of reinforcement. The high grade of the concrete was shown by the average ultimate tensile resistance of 330 pounds developed by specimens 7 months old, that being the age of the cylinder when first subjected to pressure. Under a head of 28 feet the leakage from the cylinder was over 900 gallons per hour. After the interior surface of the concrete had been grouted twice and coated with cement mortar the leakage was reduced to about 45 gals. per hour under a head of 34 feet. Under this last condition the steel was under a unit tension of only 1,100 pounds. The writer does not know what conclusions were formed from these tests, but assumes them to be reflected in the practice of the Board of Water Supply of limiting the unit stress in the reinforcement of water retaining structures to 8,000 lbs.

The existing holder tanks and reservoirs, conforming substantially to types No. 2 and 3 in design, have maximum wall thickness of 18 inches for No. 2 and 10 inches for No. 3. A considerable increase in this wall thickness would doubtless prevent seepage, but this would lead us in the direction of type No. 1.

Tanks of these types cannot be made tightly waterproofing, for the reason that such coatings to be effective must be applied to the inner surface while the wall is under tension. If applied to the outside it will be useless. Such tanks have been made tight by charging the water in them with cement, but there is no certainty of this process being successful unless all of the water in the tank be converted into grout.

In some very important particulars concrete tanks of any type are less desirable than those of brick. For, while brickwork has been discussed as an uncertain material for resisting tension, concrete is even less dependable when thus employed. The structural principles of concrete and brick masonry are essentially the same, but the latter represents the systematic assemblage of the elements. This process permits complete inspection as the work proceeds. Brick masonry does not require the exercise of any peculiar skill nor that momentary vigilance so necessary in concrete work, hence good results are more easily secured. Concrete work may turn out unintentionally bad, but it will never be accidentally good. Brickwork is not liable to the serious cracks that develop so frequently in concrete as a result of the inherent shrinkage stresses. Furthermore it is a simple matter to repair the wall of a brick tank and obtain the original strength. With concrete repairing is difficult and the results doubtful. In the event of a destructive storm during the building of a masonry tank any damage sustained by brickwork would be palpable, while impaired concrete might escape observation.

When a more extended use of concrete began 10 years ago it was believed that structures built of sand, cement and broken stone were practically immortal. They were said to become a part of geology. Doubtless the experience accrued in the last 10 years has convinced engineers that concrete structures are not more enduring than the other works of man.

In no other fields of human industry have science and skill been so extensively and successfully applied as in the production of structural steel. Manufactured from the most important of the metallic elements, the original granular formation is readily transformed into an elastic fibrous structure. Structural steel is manufactured in several grades, each differing from the other in physical properties. The desired properties of tenacity and ductility are developed by mechanical processes and by varying the proportion of certain chemical constituents.

The custom of specifying these chemical properties, in hundredths per cent., indicates the perfection the art has reached. The certainty and uniformity with which any special grade or quality of steel may be produced are in marked contrast to the random results from the manufacture of other structural materials.

An ideal stress condition may be described as that condition which is developed in a material of definite and uniform elastic properties by a completely known force. Such ideal conditions are never more nearly attained in practice than when steel is employed to resist fluid pressure.

The usual specification for steel plates for holder tanks permits a variation of 10,000 pounds in the ultimate tensile strength, and requires the elastic limit to be not less than ½ the ultimate. The ultimate strength of tank plates is generally assumed at 60,000 pounds, and records available to the writer of tests made upon thousands of tons of such material show rare instances where the variation from that figure was 5 per cent. Hence it may be said that, while steel tanks are designed upon an average resistance which varies 10 per

cent., masonry tanks are proportional upon the basis of the poorest material that human frailty may permit to enter the structure. Some one has defined a factor of safety as "the factor of ignorance." In the case of masonry tanks, it would be very properly defined as "the factor of our helplessness," for we are helpless to either influence or control the tensile resistance of masonry within 100 per cent.

The shell of a steel tank, like the wall of a masonry tank, receives material assistance from the bottom, though this fact is not usually considered in designing. Fig. 9 is an exaggerated representation of

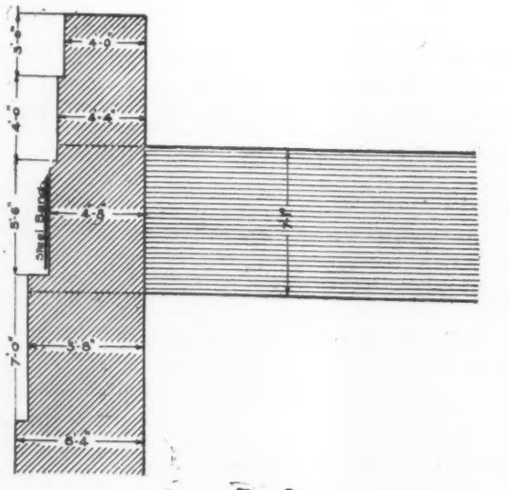

Fig. 8)

Fig. 9.

the elastic deformation of a steel tank. The thickness of the plates at the top of the shell is governed by practical considerations and is always much more than the stress conditions would require. This excess of metal, assisted by the circular girder, reduces the diametral expansion at the top of the tank to a very small amount. At the bottom of the shell the diametral expansion is reduced to an almost imperceptible increment by the restraint of the bottom. The amount of this restraint is best realized by conceiving a large tank to be cut into two parts by a vertical diametral plant. Then if the coefficient of the static friction between the bottom and the foundation equals 0.3, the friction would exceed the whole bursting pressure, tending to spread the two halves of the tank apart. Or, ignoring the friction and assuming a joint efficiency of 50 per cent. for the bottom seams, the bottom plates would be capable of resisting the entire hydrostatic pressure acting on the lowest 10 feet of the tank shell. The flexibility of the plates will, however, prevent the restraint from being materially effective above the first rim.

The assistance contributed by the bottom of the tank to the lowest rim of sides plates is much more effective when the curb angle is placed on the outside of the tank. If, with such a condition, the thickness of the lowest rim be proportioned to a stress computed from the condition of an unrestrained ring under hydrostatic pressure, an analysis of the elastic behavior of the rim in connection with the curb angle and tank bottom will show that the actual unit stress at the bottom of the rim does not exceed 10 per cent. of that assumed in designing. If the rim approximates 4 feet in height the unit stress at its upper edge will not exceed 90 per cent. of that assumed in the design. From this it will be seen that, having determined the unit stress permissible in the plates, that stress will be more nearly approached if the thickness of each rim were proportionate to the tension at the center of the rim. The frequent practice of proportioning the plate thickness to the tension at the bottom of the rim is inaccurate and results in an unnecessary amount of metal. It is certainly neither rational nor intelligent designing to accept one unit stress as permissible and then assume fictitious conditions that produce some other stress.

Though holder tanks of steel have been in use for more than 20 years there is still a lingering distrust of their security. Those that entertain this distrust seek to justify it not by the failures, but by the lack of them. The idea seems to be that a failure is more or less equivalent to a test to destruction and hence, if more steel holder tanks had collapsed, their behavior under pressure would be better understood. There is doubtless some logic in this argument, but from the fact that of all the many large steel holder tanks that have been constructed in the last 20-odd years there is a record of only one failure, we may safely conclude that a more exact knowledge of the true stresses in the tank shell would result in less metal and rivets than are employed in the practice of the present time. In connection with the recent design and construction of two large steel holder tanks, tests of prepared riveted specimen joints were made by the builder. The tests indicate:

1. That the strength of a double, butt-riveted joint is not less than the total strength of the elements of which it is composed, taken separately.

2. That the ultimate resistance of a tank plate is less than that stress which would be necessary to break down the friction in the joint.

3. That the frictional grip in such joint is so great that within the limits of working stresses there is neither shear in the rivets nor bearing stress against the shank.

Fig. 10 is a partial cross-section of the spherical type of tank o

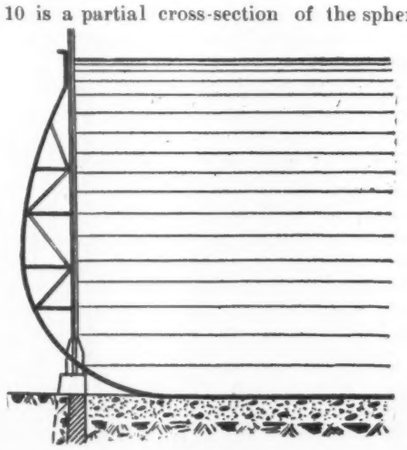

Fig. 10.

which several have recently been built in Europe. With this design the thickness of the plating is independent of the diameter. The profile of the tank on a radial line is an equilibrium curve known as the hydrostatic chord. The tension at any point in the walls of any vessel having a curved perimeter under fluid pressure in equilibrium is equal to the product of the unit pressure and the radius of curvature. Hence, as the internal pressure is met by the tensional resistance in a meridional plane without developing any serious circumferential tension, it is evident that this design permits the plating to be very thin. To make the nearest approach to ideal stress conditions requires that the shell of this tank shall be uniform, elastic and unrestained.

The advocates of this new type of tank are asserting its alleged advantages with an immoderation that may remind us of the enthusiasm with which the suspension bunker was first introduced. The suspension bunker has been in the field now for some 20 years. It has its merits and is extensively used, but it has absolutely failed to eliminate those standard types of bin construction that were in use before it appeared. If a spherical tank is to eliminate the usual cylindrical form, the design must be demonstrated as possessing more considerable advantages than have yet been shown.

The *Journal of Gas Lighting*, July 5, 1910, publishes a discussion of steel holder tanks by M. Edouard Bonnet. The paper is an argument for the new type of tank, and consists in comparing the alleged disadvantages of the cylindrical construction with the supposed advantages of the spherical tank. Nothing is stated by the author about the disadvantages of the spherical tank.

M. Bonnet begins his condemnation of cylindrical tanks by acknowledging the bottom plates as being necessary to retain the water. In the next sentence he states that the bottom contains an "amount of metal which has no really useful place in the construction." If that statement be true it follows that the shell of the tank is also unnecessary, as likewise its only purpose is to retain the water.

The thickness and behavior of the tank shell are next discussed by M. Bonnet. He takes a serious view of the difficulty and labor involved in assembling the heavy plates required for the cylindrical tank. The fact that the entire first rim of the 10,000,000 holder, at Newtown, was assembled and riveted in 11 working days without ex-

traordinary effort would suggest that M. Bonnet is not without prejudice. He then states that "While the interior pressure increases the diameter of the tank, the bottom of the lower ring of plates, strengthened by the angle iron and held by the bottom plates, cannot take part in the general tension of the circumference, and causes partial strains which are dangerous and lead to bursts." Can M. Bonnet explain why the condition of the lower ring of plates is dangerous when it is "strengthened by the angle iron and held by the bottom plates?" Also, why are "partial strains" more serious than the entire strain?

His next statement is that the circular girder at the top of the tank does nothing. "Its only effect is to insure the circular form of the tank and it adds nothing to its strength." From this we may assume that a tank is not strengthened by insuring its circular form. It would also greatly enlarge our knowledge of those "certain parts" of a cylindrical tank in which the metal "exerts itself badly" were definitely stated.

A fair comparison of the two types of tank will show that the reduction in plate thickness which the spherical tank permits is its sole advantage. The spherical tank, as constructed, makes no closer approach to the ideal stress conditions upon which the design is based than does the cylindrical tank. A mechanical analysis of the hydrostatic chord assumes the element to be unrestrained at any point between the termini. The spherical tank is not and cannot be so constructed. The tank described by M. Bonnet has a number of radial vertical girders attached to the outer shell, there being apparently one girder under each guide frame standard and two in the space between. The bases of these girders are supported by pedestals outside of the shell resting upon the annular fouudation. Each vertical element of the tank coinciding with one of these girders is subjected to a considerable restraint, while an element midway between is free, except to that extent to which the rigidity of the plating may transmit the restraint of the girders around the circumference. Will some advocate of the spherical tank enlighten us as to what extent the ideal stress conditions are disturbed by this restraint, or as to the nature and amount of the stress thus induced? Further, M. Bonnet, referring to the stress induced in a cylindrical tank by the guide frame, states, "Such a tank has thus added to it considerable strains, varying in position and intensity, which it is almost impossible to ascertain." In view of this statement it is somewhat remarkable that he omits any discussion of the corresponding stresses in a spherical tank.

M. Bonnet discusses tank foundations in utter disregard of the fact that the shell of a cylindrical tank represents a circular girder of great depth and stiffness. The writer has seen a cylindrical tank of large size, the foundation of which had been undermined by adjacent excavation without serious results, A spherical tank is a relatively flabby construction and is absolutely dependent for its security upon an unyielding foundation. If one arc of the annular foundation were to subside, is the structure of the spherical tank adapted to bridging across the depression? An unequal settlement of cylindrical tanks has occurred with sufficient frequency to demonstrate the inherent strength of its form as a circular girder. The author also states, referring to spherical tanks, "even the bottom only supports the weight of water." What else does he assume the bottom of a cylindrical tank to be carrying? He then says that "calculation has shown that an important depression in this part of the foundations would require only a slight increase in the work done by the plates." We might observe in this connection calculation has shown that an important depression in this part of the foundation under a cylindrical tank would reduce the work done by the plates in the shell, and actual experience has demonstrated that no harm results. An unmistakable advantage of the cylindrical tank is its security when supported upon an annular foundation. The Consolidated Gas Company, of New York, has a holder, the steel tank of which is 193 feet 2 inches in diameter and 41 feet 9 inches deep, the tank being supported by an annular pile foundation. The site was originally under water and was filled in upon black river mud of a considerable depth. The foundation has now carried the load for 22 years without settlement.

All innovations should receive fair hearing and full consideration, but, on the other hand, the claims of merit should be candidly and honestly presented. The writer does not believe that much can be accomplished for an alleged improvement by subjecting existing practice to unfair and truthless criticism. Though steel tanks have been built in large numbers, there seems to have been little investigation into the behavior of the inner surface of the plating where exposed to the action of the water in the tank. Hence the question has been frequently raised as to what the effect of the water upon the inner surface of the shell might be. During the present year the writer had opportunity to examine a large steel tank, as built at a time when the manufacture of open hearth steel plates was in its infancy. The material in this tank has been shown to be of very inferior quality, but, notwithstanding this fact, after nearly 20 years' exposure to the action of the water, the interior surface of the shell could be described as being in perfect condition.

The Consolidated Gas Company has usua lly placed its large steel tanks with some part of their depth below the level of the surrounding ground. It has been its practice to protect the shell below the ground level with a thin concrete wall. A recent examination of the tanks so located showed that the concrete remained in absolute contact with the surface of the plates and that no corrosion had occured. Among the distinct advantages of steel tanks, when compared with those of masonry, are:

1. Less cost.
2. Shorter period required for construction.
3. The ease with which the quality of the work may be controlled during construction.
4. The high state of development in fabrication and erection.
5. Greater reliability under stress.
6. Accessibility for inspection.
7. Tank may be placed at any elevation with relation to the ground line that may be desired.
8. No liability to damage by storm during construction.
9. No internal stresses from shrinkage or temperatures that are serious.
10. Possibility of rectifying an unequal settlement.
11. No liability to cracking from undetermined causes.
12. The ease of making repairs and obtaining the original strength.

The writer may be charged with entertaining a strong prejudice against masonry tanks. He will admit that he is opposed to any type of tank, 25 to 40 per cent. of the cost of which may go into digging a hole in the ground instead of putting quality into the structure.

The Woodall-Parkinson Patent Expansion Nipple.

The use of steel tubes in place of cast iron pipes has been much retarded owing to the difficulty of making a sound connection to main. This difficulty has now been overcome consequent upon the introduction of the ingenious invention known as the Woodall-Parkinson "Patent Expansion Nipple," the main features of which are shown in our advertisement columns. And this device has removed many of the long-standing objections to the general adoption of steel tubes.

The nipple is made exclusively from specially drawn steel tubes, and forms a joint with the main equal in strength, soundness and permanence to the branch of a tee piece, and connections can be made at any angle to the main, while the cost is small compared with that of the clips, bands or other devices hitherto employed. This device rivets steel to steel by expanding the end of the branch connecter inside the main, and the joint has been tested under a pressure exceeding 2,000 pounds per square inch. This system has been largely adopted in all parts of the world for making connections to steel mains, due to its great strength, simplicity, soundness and durability. It, therefore, follows that, owing to the introduction of this device, the demand for steel mains will rapidly increase.

Interesting Legal Opinions

RESPECTING GAS PRACTICE.

[Prepared for the JOURNAL, by MR. JOHN EDSON BRADY.]

Gas Company Liable for Injuries to Oyster Beds by Water Gas, Tar and Other Injurious Substances.—An important decision, bearing on the liability of a gas company for injuries to oyster beds, caused by refuse thrown by the Company into the waters where the beds are located, was recently handed down by the Supreme Court of Rhode Island. The action was one of trespass, brought by the plaintiffs against the defendant Company to recover damages for injuries to their oysters and quahaugs, commonly known as clams, and for the expense of cleaning the damaged shell-fish, made necessary by the deposit, in the Providence river, of tars, oils and other deleterious substances used by the defendant in the manufacture of illuminating gas. The tar, oils, etc., were carried by the currents

and tides to the oyster beds and tide-flowed lands of the plaintiffs, situated about two miles below the defendant's gas plant.

It appeared from the evidence that the winter of 1903-4 was a particularly severe one, and that the upper parts of the bay and the river were frozen over in January, 1904, and remained so for some eight weeks or so, during which time the plaintiffs, as well as other oyster men, were prevented from taking oysters from the oyster beds on account of the freezing of the river. At the time they left off work in January, 1904, the oysters and little-necks were apparently in good marketable condition; but immediately upon resuming the work of catching oysters and little-necks, in March and April, 1904, after the ice had broken up, some of the shellfish were found to be dying, and the flavor of those which were not dying was so affected as to render them unmarketable. They had the peculiar flavor of gas, which is pungent, disagreeable and so repellant to the taste as to make them uneatable. The escape of the water gas tar and refuse matter into the river continued throughout the years of 1904 and 1905 and caused more or less damage to the shellfish during both years.

The effect of the water gas tar upon the oysters and quahaugs was described by the court in the following language:

"Water gas tar is a peculiar and distinct chemical substance which is a by-product of the manufacture of water gas. Water gas, which of itself is non-luminous, requires the injection into it of carbons, and this is done by the use of a low grade of petroleum which is passed into a chamber containing the gas at a high temperature, where it is decomposed into a gas and mixed with the water gas, and thereby supplies the necessary particles of carbon to cause light when the gas is burned. From this process there is a refuse which is collected when the gas is cleansed and cooled which is known as 'water gas tar.' This substance has practically the same density which salt water of the upper waters of Narragansett Bay has. It will, therefore, float in suspense in the water for long distances before it settles. It was easily carried by the tides and currents to the plaintiffs' beds and even farther south. Water gas tar is of a viscous or sticky nature. It is repellant to the nature of the oyster, which will not admit it, and while this is present in the water in any quantity the oyster will not feed, and will therefore eventually starve itself to death. It also kills the vegetable organism in the water which naturally exists in large amounts, and which furnishes the chief food upon which the oyster and other shellfish live. By this lack of food the oyster is starved and destroyed."

The Company raised various questions as to the right of the plaintiffs to maintain the oyster beds at the place where they were located. These questions were all decided favorable to the fisherman. There was no dispute as to the marketable value of the oysters and clams, nor was it disputed that the ratio of deaths among the oysters and little-necks was from 50 to 75 per cent.

The Company was held liable upon the theory that its acts constituted a nuisance. To quote from the opinion of the Court:

"An examination of the transcript of testimony suffices to convince us that there is a strong preponderance of evidence in favor of the plaintiffs upon their contention that the oysters and quahaugs were injured and killed by water gas tar and kindred products manufactured by the defendant and by it allowed to escape from its premises into the Providence river and public waters of the State. The defendant is not required by law to manufacture water gas or any other gas. It is permitted to manufacture either, for its own emolument and profit, but with the implied, if not express, limitation that it shall not become a public or private nuisance, that if it creates deleterious and poisonous substances it will safely dispose of the same without injury to others, and any manufacturer who allows his deleterious waste product to contaminate the waters of the State, be they public or private, is liable to any person who is injured thereby in his private capacity and apart from being merely one of the public, provided he can trace to its origin the noxious substance whereby he is damaged. This the plaintiffs have done. The damage was proved to have been occasioned by water gas tar coming from the premises of the defendant. They make and must confine and safely keep, this and other dangerous and deadly products of their manufacture at their peril."

Since the time when the circumstances, on which this action was based, transpired, a statute has been passed in Rhode Island, the object of which is to protect shellfish from injury by refuse of the manufacture of gas. Gen. Laws 1909, c. 144, § 6, reads as follows: "Sec. 6. No person or corporation shall deposit in, or allow to escape into, or shall cause or permit to be deposited in, or allowed to escape into, the waters of Providence or Warren harbors or Providence or Warren rivers, any material used in connection with, or product of, the manufacture of gas, which may cause disagreeable odors or defile the surface of vessels, boats, or other property, or the shores of said Providence or Warren harbors or rivers, or injure the healthy growth of fish or shellfish in said waters. Every person or corporation violating the provisions of this section shall be fined for each offence $100, one-half thereof to the use of the State and one-half thereof to the use of the complainant. It shall be the duty of the Harbor Commissioners to prosecute cases under complaints brought in accordance with this section." Payne & Butler *v.* Providence Gas Company; 77 Atl. Rep., 145.

Small Plant Failures and a Cause for Them.

By Mr. Charles T. Johnson, in *Elec. World.*

Electric light plants in towns of less than 4,000 or 5,000 people seem, as a class, to have acquired an unsavory reputation with bond houses and investment companies in general. Application for funds by these small plants usually brings the reply that the investing company deals only with properties having a gross revenue of more than $30,000 or $50,000 per year.

Investigation into the past history of the small companies shows the principal cause for their bad financial reputation to be lack of appreciation by the owners, usually business men in the small town, of the requisites of a good electric plant manager. In many instances the owners seem to have regarded a man's ability to run an engine and to string wires in houses as the determining requirements of a manager or superintendent. The position appears to be worth not more than $60 or $70 per month. As a result, many plants have been subjected to a $60 per month quality of management and are much the worse for it. The net earnings were not such as to encourage the stockholders to put in additional investment to take care of increasing demand or to replace obsolete machinery, and consequently the finances of the company got into a worse, instead of a better shape.

In not a few instances all of the difficulties of small companies can be traced to cheap and inefficient management. The superintendent of a small plant should be a broad-minded, diplomatic and technically trained man. His services should be worth at least $100 per month. A man that is not worth this amount is not of the type to be put in charge of an investment of $15,000 and upward. He should be capable of taking the place of any one of his employees in cases of emergency and should be willing and able to work 36 hours at a time if occasion require it. He should also be a man with a pleasing personality and one capable of getting into and staying in the good graces of the town board or city council. He should have enough technical knowledge to enable him to know when his plant is being operated economically, to test meters, to figure line losses and to take care of the many problems he meets which require for their solution a working knowledge of practical electrical engineering.

When the necessary money is put at the disposal of a superintendent of this caliber he can, in almost every case, turn a losing proposition into a paying one. Practically all of the well-managed plants in small towns are on a good financial basis, and it is safe to assume that the right man is not in charge, or the man is not supplied with sufficient money to get the plant started right, when we see a losing one.

Recent Patent Issues.

Prepared for the American Gas Light Journal by Royal E. Burnham, solicitor of patents and counselor in patent causes, 833 Bond Building, Washington, D. C., from whom illustrations and specifications of any patent mentioned below may be obtained for 10 cents.

974,260. Automatic Pressure Regulator for Gas Collecting Mains. O. F. Greim, Milwaukee, Wis., assignor to the Milwaukee Coke and Gas Company, same place.

974,551. Gas Economizer and Purifier. J. W. Carpenter, Salem, W. Va.

974,555. Regulator for Gas Generating and Supply Systems. F. L. Cross, Detroit, Mich.

974,565. Automatic Gas Igniter. E. E. Gerald, Baltimore, Md.

964,639. Gas Generator. E. Brauss, Linden, Germany, assignor to G. Korting, Aktien-Gesellschaft, Kortingsdorf, Germany.

974,650. Top for Meter Boxes. E. H. Ford, Hartford City, Ind.

974,651. Means for Setting Meters. E. H. Ford, Hartford City, Ind.

974,671. Gas Burner. C. A. Johnson, Bucklin, Mo., assignor of one-fourth to J. T. Coleman, Tolar, N. M.

974,748. Automatic Gas Lamp. W. A. Carr, Springfield, Mass., assignor to C. H. Tenney, Hartford, Conn.

Meeting Times

OF THE VARIOUS GAS ASSOCIATIONS.

American Gas Institute.—Annual meeting, October, 1911. ————. Officers: President, Donald McDonald, Louisville, Ky. Secretary, A. B. Beadle, 29 West 39th st., N. Y. City.

Canadian Gas Association.—Annual meeting, June, 1911. ————. Officers President, Arthur Hewitt, Toronto, Ont.; Secretary and Treasurer, John Keillor, Hamilton, Ont.

Empire State Gas and Electric Association.—Annual meeting, Nov., 1910. New York City. Officers: President, C. R. Huntley, Buffalo, N. Y.; Secretary, C. H. B. Chapin, 29 W. 39th street. New York City.

Guild of Gas Managers of New England.—Annual meeting, December. Young's Hotel, Boston; monthly meeting, second Saturday. Officers: President, A. K. Quinn; Newport, R. I.; Secretary, Walter G. Africa, Manchester, N. H.

Illinois Gas Association.—Annual meeting, time, March, Chicago, Ills. Officers: President, A. S. Harrington, Chicago, Ill.; Secretary-Treasurer, C. B. Strohn, Elgin, Ill.

Illuminating Engineering Society.—Annual meeting, Oct., —, 1911. Meetings of Sections, monthly. President, E. P. Hyde, Cleveland, O.; Secretary, Preston S. Millar, 29 W. 39th street, N. Y. City. Sections: New York: Secretary, Albert J. Marshall, 36 West 39th street. New England, Secretary, L. D. Gibbs 39 Boylston street, Boston, Mass. Philadelphia, Secretary, F. N. Morton, Broad and Arch streets Chicago, Secretary, F. H. Bernhard, Marquette Building.

Indiana Gas Association.—Annual meeting, January 19, 1911, Fort Wayne, Ind. Officers: President, Carl H. Graf, Indianapolis; Vice-President, S. E. Mulholland; Secretary-Treasurer, Philmer Eves, Indianapolis Gas Company, Indianapolis, Ind.

Iowa District Gas Association.—Annual meeting, time, June, 1911, ————. Officers: President, Austin Burt, Waterloo, Ia.; Secretary and Treasurer, G. I. Vincent, Des Moines, Ia.

Kansas Gas, Water and Electric Light Association.—Annual meeting, time, ———— Wichita, Kas. Officers: President, W. A. Scothorn, Hutchinson, Kas.; Secretary and Treasurer, J. D. Nicholson, Newton, Kas.

Michigan Gas Association—Annual meeting, time, Sept. 1911; Detroit, Mich. Officers: President, A. P. Ewing, Detroit, Mich.; Secretary-Treasurer, Glenn R. Chamberlain, Grand Rapids, Mich.

Missouri Electric Light, Gas, Water Works and Street Railway Association.—Annual meeting, April 13, 14 and 15, 1911; St. Louis, Mo. Officers: President, R. J. Irvine; Secretary and Treasurer, W. J. Cunningham, Springfield, Mo.

National Commercial Gas Association.—Annual meeting, December 5-13, 1910. Boston. Officers: President, E. N. Wrightington, Boston; Secretary, Louis Stotz, 29 West 39th street, New York City.

Natural Gas Association.—Annual meeting, May 16, 17 and 18, 1911; Pittsburgh, Pa. Officers: President, John M. Garard, Columbus, O, Secretary, T. C. Jones, Delaware, O. Editor Wrinkle Department, F. W. Stone, Ashtabula, O.

New England Gas Association.—Annual meeting, third Wednesday in February; Boston. Officers: President, W. H. Snow, Holyoke, Mass.; Secretary-Treasurer, N. W. Gifford, East Boston, Mass.

Oklahoma Public Utilities Association.—Annual meeting, May 9, 10 and 11, 1911. Officers: President, E. C. Reynolds; Secretary, Galen Crow, Guthrie, Okla.

Pacific Coast Gas Association.—Annual meeting, Sept. 20, 21 and 22, 1911, Oakland, Cal. Officers: President, F. A. Leach, Jr., Oakland, Cal.; Vice-President, W. Baurhyte Los Angeles, Cal.; Secretary-Treasurer, John A. Britton, 925 Franklin street, San Francisco, Cal.

Pennsylvania Gas Association.—Annual meeting, April, 1911, Reading, Pa. Officers President, J. H. Keppelman, Reading, Pa.; Secretary-Treasurer, William H. Merritt Lebanon, Pa.

Society of Gas Lighting.—Annual meeting, December, 1910; monthly meeting, second Thursday. Place, New York City. Officers: President, Fred. S. Benson; Secretary, George G. Ramsdell, Waterbury, Conn.

Southern Gas Association.—Annual meeting, April 19-21, 1911, Montgomery, Ala. Officers: President, H. W. Frund, Salisbury, N. C.; Secretary-Treasurer, Jas. Ferrier, Rome, Ga.

Southwestern Electrical and Gas Association.—Annual meeting. May, 1911, ———— Tex. Officers: President, W. B. Tuttle, San Antonio, Tex.; Secretary, H. B. Head, Stephensville, Tex.

Wisconsin Gas Association.—Annual meeting, May ————, 1911, Milwaukee, Wis. Officers: President, W. H. Winslow, Superior, Wis.; Secretary-Treasurer, Henry Harmon, Milwaukee, Wis.

Items of Interest

FROM VARIOUS LOCALITIES.

THE McBain-Bush proprietorship of the Brockport (N. Y.) Gas Light Company is at end, the property having been sold by them to a client of Mr. E. E. Witherby.

THE Massachusetts State Board of Gas and Electric Lighting Commissioners has indorsed the plan of merger proposed for the amalgamation into the Grafton Electric Company of the electric companies now doing business in Uxbridge and Northbridge, and Douglas, Upton and Milbury.

JUDGE GEORGE R. GRAY, of the New Jersey Court of Errors and Appeals, died at his home, 30 Franklin street, Newark, N. J., the morning of the 4th inst. He had been Vice-President of the Essex and Hudson Gas Company, which Company is now a factor in the properties of the New Jersey Public Service Corporation. He was in his 69th year, having been born in Newton, N. J., March 22, 1842. He was one of the best corporation law authorities in his native State, and traced his lineage in direct succession to the historic Gray family, of Delaware.

MR. T. M. LEWIS, of St. Louis, Mo., informs us that the proprietors of the Monticello Electric Light Company, who own the right under franchise to supply gas in Monticello, which by-the-way is a thriving town in Iowa, will shortly be in the market for bids from constructors of gas plants. The concern is in every good sense responsible for its agreements.

THE Berwick (Pa.) Consolidated Gas Company, by order of Judge Archibald, of the United States Circuit Court, was sold by the Receivers (Messrs. W. E. Barrett and S. S. Jayne) at public sale, as per announcement. The purchaser was a committee representing the bondholders, and the successful bid was $40,700.

MESSRS. R. ROSS PERRY & SON, who have been attorneys for the Washington (D. C.) Gas Light Company for a number of years, have ceased to act in such capacity. It is likely that their successors will be Mr. Wilton J. Lambert and Mr. Benjamin S. Minor.

THE Committee on Light, Richmond (Va.) City Councils, has notified the City Engineer to include in the specifications for the new concrete bridge, to span the James river, a duct for a gas main, to be 20-inches in diameter. The main is intended for the better supply of gas to and through the Richmond southside district.

THE piping of natural gas from Louisiana to points outside the State is to be made a matter for the State Conservation Commission.

THE Pintsch Compressing Company proposes to build a plant in River street, Brambleton Ward, Norfolk, Va., for the charging of the tanks used as storage reservoirs for the lighting of railroad coaches of the Southern Railroad System.

DURING the last quarter (ended Sept. 30), the treasury of Providence, R. I., earned these sums from the operation of royalty tax on franchise payments on gas and electric accounts:

	Gross. 1910.	Tax.
Providence Gas Company.....	$233,305.99	$6,999.21
Narragansett Electric Company.	203,556.27	10,227.81
Last Year.		
Providence Gas Company.....	$215,850.52	$6,475.52
Narragansett Electric Company.	201,300.08	10,068.65

AT last the efforts of the Commercial Club to enlist capital in the operating of a gas plant at Coeur d'Alene, Idaho, seem to be successful. At any rate at the Club's meeting the last week in October, Mr. Ray M. Hart, of Spirit Lake, informed the members that, with their assistance, he thought his application to the Council for a 50-year franchise on the account named would be favorably considered.

THE style in which the new show rooms of the Cedar Rapids (Ia.) Gas Company are fitted up has actually amazed the quiet citizens of that virtuous settlement. They are "flocking" to inspect the show, however, and to absorb the clever arguments of the "sales-persons" in respect af the virtues and other values of gas as an active agent in the home—particularly in respect of the kitchen.

SOME time ago Judge Prest issued a temporary restraining order by enjoining the Taylor Light, Fuel and Power Company from shutting off the gas supply, advancing the rate or removing its pipe line in Gas City, Kas., which is perhaps 1 mile east of Iola. This injunction was issued immediately after the announcement by the local company that it would put in a meter system, adjusting the selling price at 75 cents per 1,000.

DEAR JOURNAL: I inclose you the following mention, which likely will be of interest to many of your readers, to whom the subject no doubt is known: "Mr. E. W. Eustace, for the past 10 years Superintendent of the gas distribution division of the Peoria (Ills.) Gas and

Electric Company, recently resigned therefrom—in fact, the resignation became effective the 1st inst. Towards the close of October, in appreciation of the high esteem in which he was held by his associates, they banded together and purchased a handsome Elgin gold watch, which was presented to him in the office of the Peoria Company the afternoon of the 29th ult. The presentation was made by State Representative Thomas N. Gorman, who in well selected speech told Mr. Eustace what they thought of him. The recipient, taken completely by surprise, managed eventually to express his thanks.—R. R. M."

At a meeting, held in Chicago, the 1st inst., these officers were re-elected to conduct the affairs of the Du Quoin (Ills.) Light, Heat and Power Company: President, Henry I. Lea; Vice-President, Frank W. Hanwell; Treasurer and Secretary, Frank M. Elliot.

The E. E. Witherby Company has completed negotiations under which it will proceed with the construction of a gas distributing system at Westbrook or South Portland, Me. Proper contracts have been made under which the two named settlements will be supplied with gas purchased from the Portland Gas Light Company.

"D. R. M.," writing from Chicago under date of the 3d inst. among other things says: "Enclosed find * * * to cover my 20th annual subscription to the Journal. The regular fortnightly meeting of the Branch Store Managers of the Peoples Gas Light and Coke Company was held the evening of the 28th ult., and a large number of the Company's representatives responded in person. The main topic for consideration was the sale and introduction of the art dome dining room fixtures. Mr. H. Jenkinson, who is an expert at interior illumination—and a bully good Englishman he is, don't you know—was the gentleman to act as salesman. It goes without saying that he handled the subject in most excellent manner. At the close of the meeting, Mr. Charles E. Walker, the genial but modest Manager of the 35th street branch store, read a paper on 'Salesmanship,' which was heartily applauded. The ideas and suggestions expressed in same are, in the writer's opinion, certainly worth studying. I inclose a copy of it. * * * Mr. John C. D. Clark, whom you all know, was present at the meeting, and, having just returned from New York, where he had been attending the meeting of the American Gas Institute, favored the boys with some very interesting remarks concerning the great Consolidated Gas Company, of your city, which were well received by those present."

Mr. E. E. Witherby, as Engineer to the South Shore Gas Company, of Long Island, N. Y., wishes to report that gas will undoubtedly be turned on from the new works about the 15th prox., the construction being in shape to warrant the prediction. The Company will start with 300 consumers—this number in a territory that has never been served with gas.

The contract for the new retort house furnishings for the Montreal Light, Heat and Power Company has been awarded to the Parker-Russell Mining and Manufacturing Company, of St. Louis. It calls for the placing of 11 benches of 12's, throughs, 22 feet 3 inches in length. All settings are to be made of a silica mixture, in special forms, to fit the retorts. The retorts, all the firebrick and silica shapes to be of the Parker-Russell type, and to be made by them at their shops in St. Louis. The contract covers benches and all ironwork for same, the foul mains, tar towers and ammonia and tar condensers, the former to be by the Kerr Murray Manufacturing Company. The governors are to be supplied by the Lloyd Construction Company, and it is agreed that the work will be in readiness for firing up by June, 1911.

Every gas man should have a copy of "Bulletin" 416, published by the "Department of the Interior, Bureau of Mines," which has for its title "Recent Development of the Producer Gas Power Plant in the United States." The compiler is Mr. R. H. Fernald, who certainly deserves much credit and praise for the intelligent manner in which he deals with his subject.

Mr. J. V. Cattron has been promoted to the position of Commercial Agent for the Luzerne County (Pa.) Gas and Electric Company. His successor to the Superintendency is Mr. J. F. Gaylord, Binghamton, N. Y.

We are indebted to a correspondent in Tacoma, Wash., for the following clipping from a recent issue of the Seattle (Wash.) *Times*: "The Seattle Lighting Company may be held responsible for the death of M. McNab at the Seattle House, Fifth avenue and Jefferson street, early yesterday morning. McNab's death was due to a slot meter, the use of which in buildings of this character is prohibited by an ordinance passed by the Council and becoming effective June 6, 1909. This ordinance, introduced at the request of A. V. Bouillon, former Superintendent of Public Utilities, makes it a misdemeanor after 6 months from the date of the approval of the ordinance to maintain any prepayment meter serving more than one room or one suite of rooms. The time for the removal of all such meters in building of this character expired on December 6 last, and Secretary H. R. Clise, of the Seattle Lighting Company, advised Superintendent of Public Utilities, A. L. Valentine, that although he had not made a personal investigation, his Company simply had failed to live up to the terms of the ordinance passed a year ago last June. He said further that immediate steps would be taken to replace all prepayment meters, regardless of their location."

Last month Mr. Daniel B. Smith, since 1887 head foreman of the plants of the Charlestown (Mass.) Gas and Electric Light Company, retired from its service. Just prior to his departure for Truro, N. S., in which settlement he will spend the remainder of his life, the works' men presented him with a fine specimen of a meerschaum pipe, the mountings being of gold.

As a result of the reconstruction of the gas plant of the Asheville (N. C.) Electric Company, the proprietors of the same have announced a reduction in the gas rate of 50 cents per 1,000 cubic feet, the concession to be operative from the 1st inst. The output of gas in Asheville shows an increase of 20 per cent. so far this year.

The proprietors of the Woonsocket (R. I.) Gas Company have purchased a site from the Manville Company whereon it is proposed to construct a gasholder up to retaining 500,000 cubic feet.

Mr. Henry B. Newman, Superintendent of the Canisteo (N. Y.) Gas Company, has been instructed to take over the additional duties attached to the management of the Hornell Gas Light Company, vice Mr. John Milne, resigned.

Mr. Wilson D. Smith, at one time prominent in the affairs of the Athol (Mass.) Gas and Electric Company, died at his home in Athol, the morning of the 3d inst. He resigned the position with the named Company to take charge of the Athol and Orange Street Railway Company.

Under the royalty agreement between the authorities of Philadelphia and the United Gas Improvement Company, the royalty on gas sold during the quarter ended September 30 last amounted to $287,470.20. This sum shows an increase of something over $15,000 for the corresponding quarter of 1909.

The Perth Amboy (N. J.) Gas Company has asked the authorities of Woodbridge for the right to lay mains through Rahway avenue to reach the outlying settlement of Chrome.

Mr. Henry B. Hoyt, General Manager of the Jacksonville (Fla.) Gas Company, has completed plans for the construction of a series of buildings, on a plot at West Church street and Seaboard Air Line Railway, to house new generating and condensing and purifying apparatus. The growth of the business at this point is really amazing.

President J. I. Blake, of the Consumers Gas Company, Toronto, Ontario, recently said that, if nothing untoward happened, the gas rate in Toronto would be down to 70 cents per 1,000 cubic feet ere the close of 1911.

The Washington (D. C.) Gas Light Company has placed on record a general mortgage (the American Security and Trust Company being the Trustee), aggregating $5,200,000, and bearing 5 per cent. interest. The effect of the issue to secure funds to pay off, and fund outstanding indebtedness and for the settlement of obligations incurred on betterment and extension accounts.

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MONDAY, NOVEMBER 14, 1910.

The Market for Gas Securities.

The political upheaval of the week does not seem to have brought with it that tremendous decrease in quoted values of standard securities which it was freely claimed by the party in power would ensue were they disturbed in their tenure of office. Four years ago we had sufficing manifestation of how quotations could be smashed, as they were smashed by a certain section of the political financiers. Consolidated closed last week at 138 to 138½, and the relative figures to-day (Friday) at noon 135½ to 135¾.

The shrinkage in Brooklyn Union is even less than that in Consolidated, the figures to-day being returned at 134, bid, as against 136 bid a week ago. Investors have seemingly in a measure not realized the higher standing of Laclede gas, for its securities show no appreciable change. Peoples, of Chicago, is lower, and Massachusetts gas remains unchanged, at the recent high record figures made in it.

Gas Stocks.

Quotations by George W. Close, Broker and Dealer in Gas Stocks.

115 BROADWAY, NEW YORK CITY.

NOVEMBER 14.

☞ All communications will receive particular attention.

☞ The following quotations are based on the par value of $100 per share:

N. Y. City Companies.	*Capital.*	*Par.*	*Bid.*	*Asked*
Consolidated Gas Co.	$78,177,000	100	135¼	135¾
Central Union Gas Co.—				
1st 5's, due 1972, J. & J.	3,000,000	1,000	98	101
Equitable Gas Light Co.—				
Con. 5's, due 1932, M. & S.	1,000,000	1,000	—	105
Mutual Gas Co.	3,600,000	100	155	165
New Amsterdam Gas Co.—				
1st Con. 5's, due 1948, J. & J.	11,000,000	1,000	100	101½
New York & Richmond Gas Co. (Staten Island)	1,500,000	100	35	50
1st Mtg. Gold Bds. 5 p. ct.	1,000,000	—	98½	100½
New York and East River—				
1st 5's, due 1944, J. & J.	3,500,000	1,000	104	107
Con. 5's, due 1945, J. & J.	1,500,000	—	97	101
Northern Union—				
1st 5's, due 1927, J. & J.	1,250,000	1,000	94	100
Standard	5,000,000	100	—	75
Preferred	5,000,000	100	70	100
1st Mtg. 5's, due 1930, M. & N.	1,500,000	1,000	100	105
The Brooklyn Union	15,000,000	1,000	134	137
1st Con. 5's, due 1948, M. & N.	15,000,000	—	106¼	107
Yonkers	999,650	500	130	—
Out-of-Town Companies.				
Bay State	50,000,000	50	⅝	¾
" Income Bonds	2,000,000	1,000	—	75
Binghampton Gas Works	450,000	100	—	65
" 1st Mtg. 5's	500,000	1,000	90	96
Boston United Gas Co.—				
1st Series S. F. Trust	7,000,000	1,000	82	85
2d " " "	3,000,000	1,000	47½	50
Buffalo City Gas Co.	5,500,000	100	5	8
Bonds, 5's	5,250,000	1,000	73	73½
Capital, Sacramento	500,000	50	—	35
Bonds (6's)	150,000	1,000	—	—
Chicago Gas Co. Guaranteed Gold Bonds	7,660,000	1,000	104	106¼
Cincinnati Gas and Electric Co.	29,500,000	100	92	93
Columbus (O.) Gas Co., 1st Mortgage Bonds	1,500,000	1,000	96	98
Columbus (O.) Gas Lt. & Heating Co.	1,682,750	100	90½	91
Preferred	3,026,500	100	75½	80
Consumers, Toronto	2,000,000	50	200	204
Consolidated, Baltimore	11,000,000	100	106½	—
Mortgages, 6's	3,600,000	—	89	100
Chesapeake, 1st 6's	1,000,000	—	—	—
Equitable, 1st 6's	910,000	—	—	—
Consolidated, 1st 5's	1,490,000	—	—	112
Consolidated Gas Co. of N. J.	1,000,000	100	15	17
Con. Mtg. 5's	880,000	1,000	92	95
Bonds	75,000	—	—	100
Denver Gas and Electric	3,500,000	—	102	105
Detroit City Gas Co	5,000,000	50	—	50
" Prior Lien 5's	4,619,000	1,000	97	100½
Detroit Gas Co., 5's	881,000	1,000	75	80
Equitable Gas & Fuel Co., Chicago, Bonds	2,000,000	1,000	—	101
Essex and Hudson Gas Co.	6,500,000	—	39	40
Fort Wayne	2,000,000	—	—	—
" Bonds	2,000,000	—	55	—
Grand Rapids Gas Light Co., 1st Mtg. 5's	1,225,000	1,000	104½	105
Hartford	750,000	25	190	200
Hudson County Gas Co., of New Jersey	10,500,000	—	112	113½
" Bonds, 5's	10,500,000	—	103	103½
Indianapolis	2,000,000	—	60	70
" Bonds, 5's	2,650,000	—	104½	105
Jackson Gas Co.	250,000	50	82	—
" 1st Mtg. 5's	290,000	1,000	97	100
Kansas City Gas Light Co., of Missouri	5,000,000	100	—	96
Bonds, 1st 5's	3,822,000	1,000	102	104
Laclede Gas Co., St. Louis	10,000,000	100	104¼	105
Preferred	2,500,005	100	91	102
Bonds	10,000,000	1,000	102½	103
Lafayette Gas Co., Ind.	1,000,000	100	—	60
Bonds	1,000,000	1,000	60	65
Louisville	2,570,000	50	143	145
Madison Gas and Electric Co.				
" 1st Mtg. 6's	350,000	1,000	106	108¾
" 6 per cent. scrip, due 1910	100,000	25	60	60½
Massachusetts Gas Companies, of Boston	25,000,000	100	89½	90
Preferred	25,000,000	100	95	95½
Montreal Gas Co., Canada	2,000,000	100	218	218¾
Nashville Gas Light Co.	1,000,000	100	110	—
Newark, N. J., Con. Gas Co.	6,000,000	—	56	58
Bonds, 6's	6,000,000	—	137	138
New Haven Gas Co.	2,000,000	25	200	—
Peoples Gas Lt. & Coke Co., Chicago	25,000,000	100	105½	106
1st Mortgage	20,100,000	1,000	—	—
2d "	2,500,000	1,000	104	—
Rochester Gas & Electric Co.	2,150,000	50	88	—
Preferred	2,150,000	50	118	—
Consolidated 5's	2,000,000	—	104½	105½
San Francisco Gas Co., Cal.	15,500,000	—	—	—
St. Joseph Gas Co.—				
1st Mtg. 5's	751,000	1,000	100	102
St. Paul Gas Light Co.	1,500,000	100	45	47
1st Mortgages, 6's	650,000	1,000	113	116
Extension, 6's	600,000	1,000	112½	115
General Mortgage, 5's	2,465,000	1,000	90	95
Syracuse Gas Co., N. Y.	1,975,000	100	50	55
Bonds	2,047,000	1,000	100	104
Washington (D. C.) Gas Co.	1,600,000	200	383½	385
1st Mortgage, 6's	600,000	—	—	—
Western Gas Co., Milwaukee	4,000,000	—	—	—
Wilmington (Del.) Gas Co.	600,000	50	361	—

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Pittsburg Meter Co., East Pittsburg, Pa. 942

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TENDERS FOR TAR.

Tenders are invited for the entire output of tar at the Vancouver and Victoria Gas Works for a period of one or two years, commencing August 4th, 1911.

Full particulars as to quantities available, loading facilities, etc., may be obtained on application to J. C. Moon, Manager, Vancouver Gas Company.

Sealed tenders, indorsed "Tenders for Tar," must reach this office not later than December 31st, 1910.

The highest or any tender not necessarily accepted.

(Signed), W. H. HAZLITT,
Purchasing Agent,
Vancouver Gas Co., Ltd.,
1848-4 VANCOUVER, B. C.

Position Wanted.

An experienced gas manager, now employed as manager, desires a similar position either with a natural or artificial gas company, in a city of 20,000 population or over. No preference as to location of city. Can leave present position on 30 days' notice. Results guaranteed.

Address, "A.,"
1843-tf Care this Journal.

Position Wanted.

Fifteen years' experience in the manufacture, distribution and construction of high and low pressure coal and water gas of various types. At present manager of a personally rebuilt plant (high pressure). **Worn-Out Western Plants** preferred. Salary first year, $1,380. If results are satisfa tory, second salary, $2,000.

1846-4 Address, "MODERN," care this Journal.

WANTED,

Position as Manager and Superintendent of Gas Plant.

Best of references given. Fifteen years' experience in gas business. Address, "F.,"
1846-tf Care this Journal.

MR. MANAGER,

Have you an opening for young man, 28 years old, experienced in distribution, construction and new business departments? Prefer city 15,000 to 30,000. A-1 record.

1849-2 Address, "H.," care this Journal.

WANTED.

By Competent Gas Man, 29 Years Old. Position as Works Superintendent or General Foreman.

Ten years' practical experience in manufacture of coal and water gas. Prefer plant with output of 100,000 to 500,000 feet per day. Can construct or reconstruct run-down property. Can handle men and get good results.

1849-1 Address, "ASSURANCE," care this Journal.

Position of Superintendent or Manager Wanted.

Qualifications as follows: Six years as engineer of gas and electric construction, two years as superintendent of manufacturing and distribution, two years estimating on property values—gas and electricity. Would prefer a gas and electric combination, but would accept either if of sufficient magnitude. Satisfactory references from present employers. Could report for duty within 15 days from date of acceptance. Address, "B. X.,"
1848-2 Care this Journal.

Position Wanted

As a Local or Supervising Manager of a Combination Gas and Electric Property or Properties, or of Independent Properties in Either Line.

Experienced in construction and operation; 35 years old; married; strictly sober, and not particular as to location.

Address, "UNIVERSAL,"
1848-2 Care this Journal.

STANDARD REDUCTION FACTORS for GASES, by Helon Brooks MacFarland, B.S., M.M.E. Price, $1.50. For Sale by **American Gas Light Journal**, 42 Pine Street, New York City.

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Position as Superintendent or Foreman of Gas Shop.

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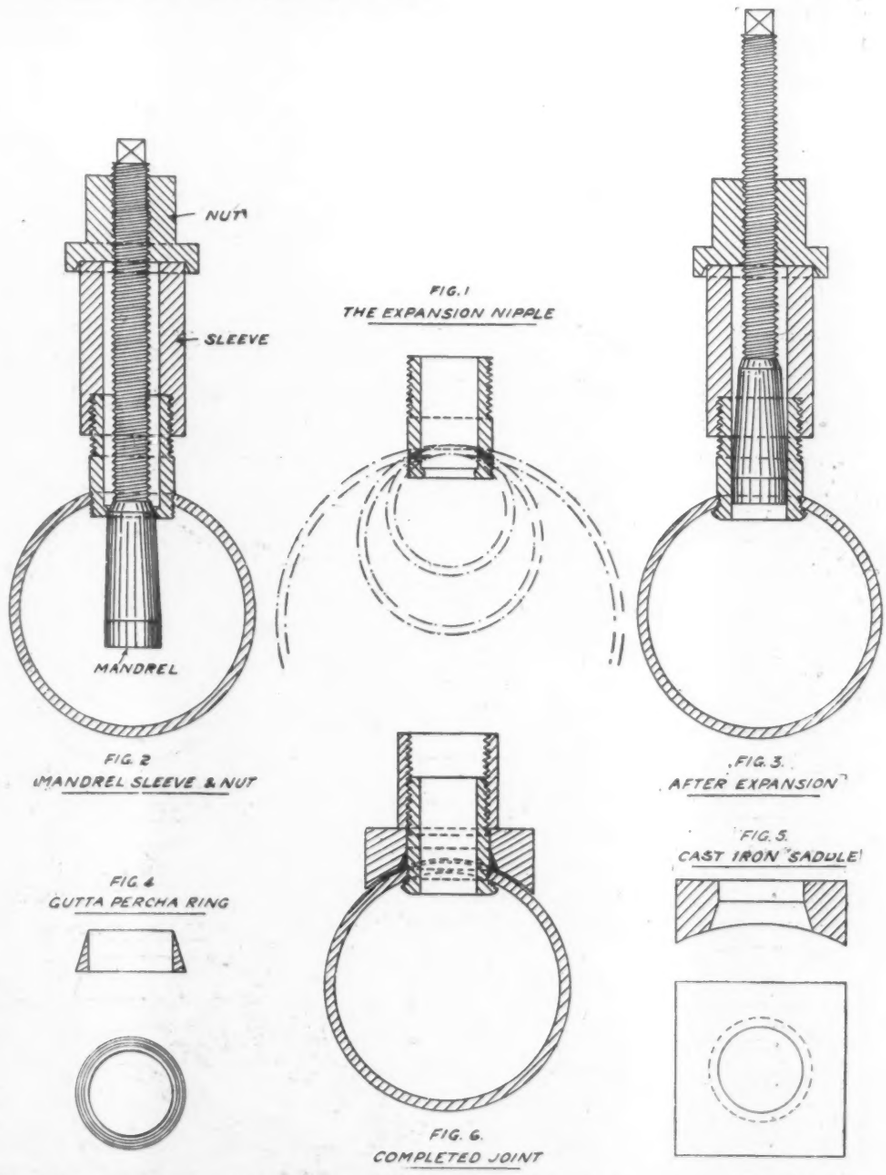

The Nipple (Fig. 1) is provided with an internal annular bead at its lower end, and on a tapered mandrel being drawn through, the bend opens out the end of the Nipple within the main. Fig. 2 shows the mandrel, sleeve and nut, which, together with a ratchet spanner, comprise the necessary tools for expanding the Nipple. These are put together and screwed into the main, the Nipple being tightened down with tongs or footprints in the usual way. Then the nut is operated with the ratchet spanner until the mandrel is drawn through clear of the expansion bead (Fig. 3), after which the sleeve can be unscrewed and the mandrel removed, leaving the Nipple firmly secured in the main. A gutta-percha ring is next placed over the Nipple and pressed down by a saddle (Fig. 5). An ordinary socket screws down on the saddle and the gutta-percha ring is squeezed into an angular section. A plug can be screwed into the socket pending the connection of the service, or a cap or elbow substituted.

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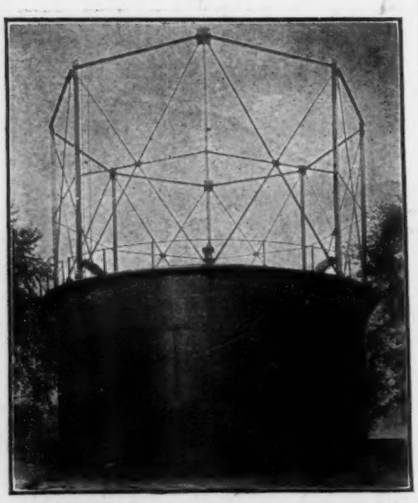

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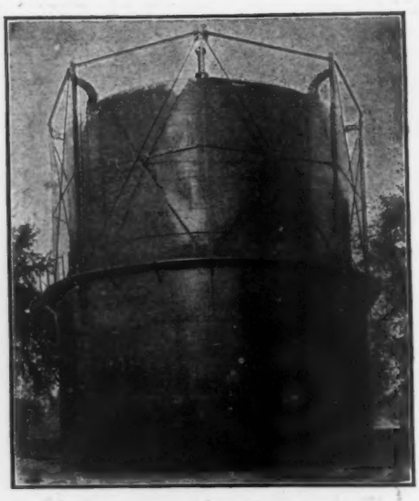

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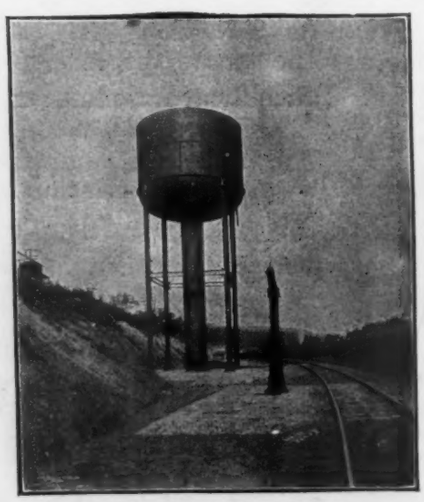

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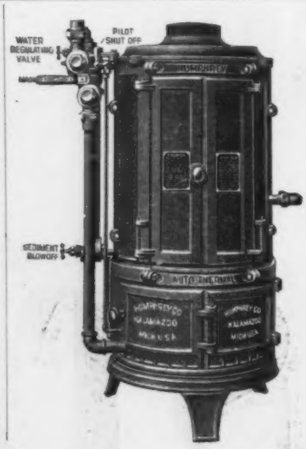

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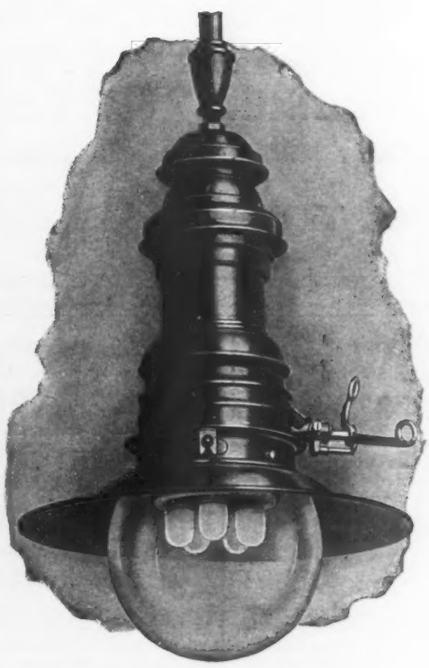

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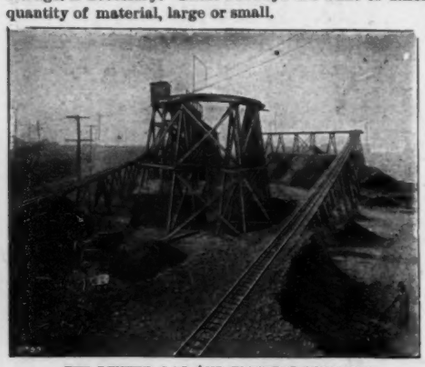

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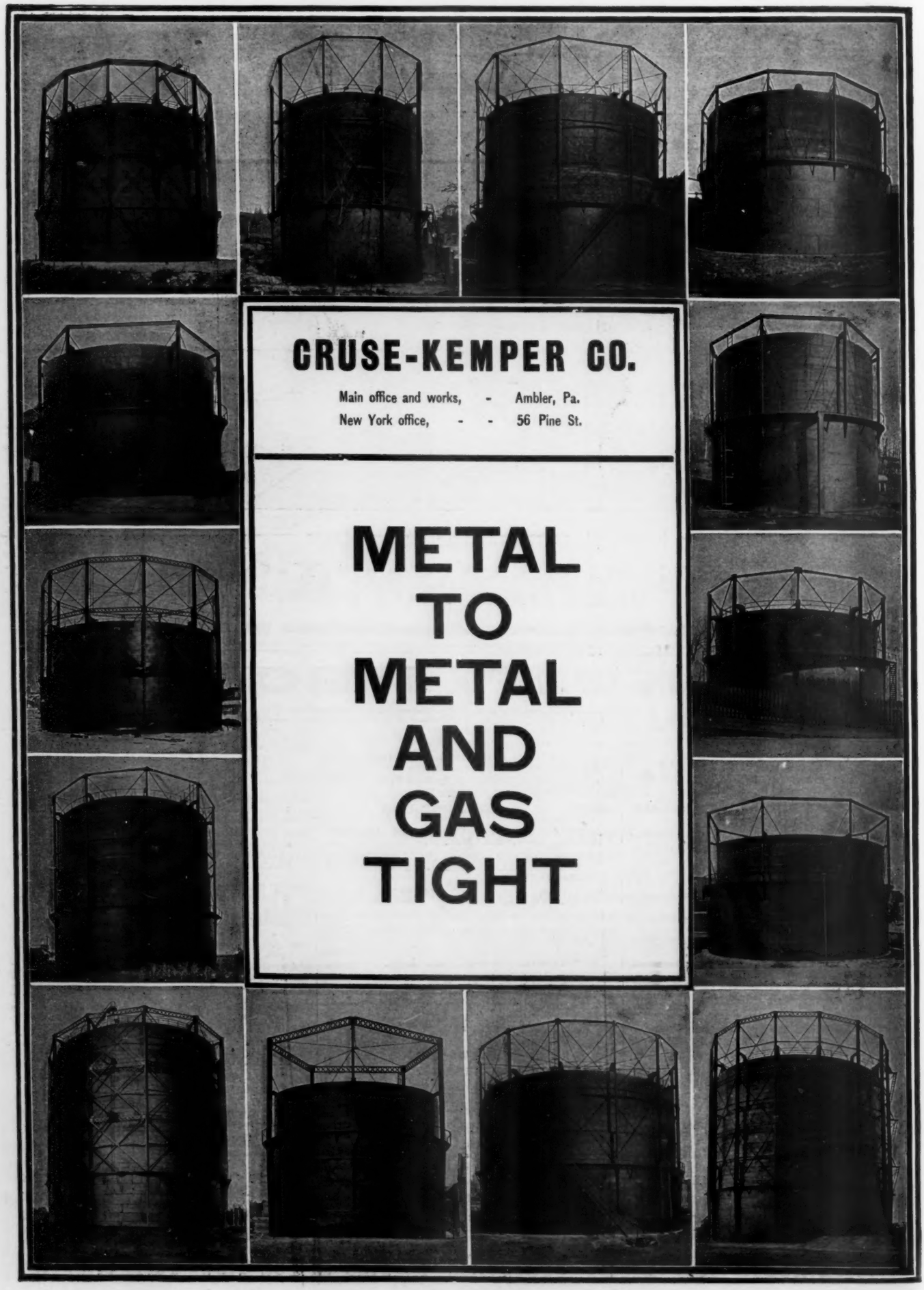
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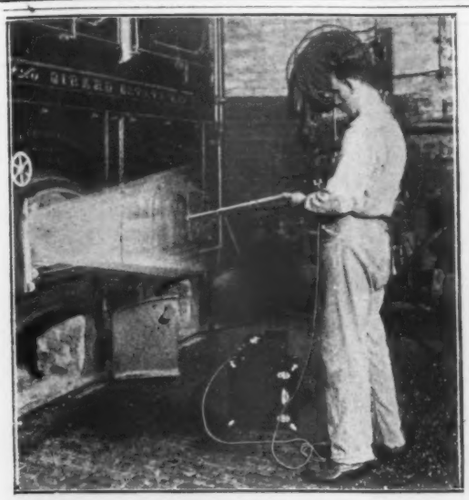

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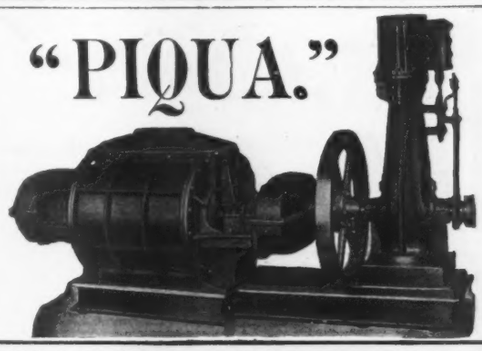

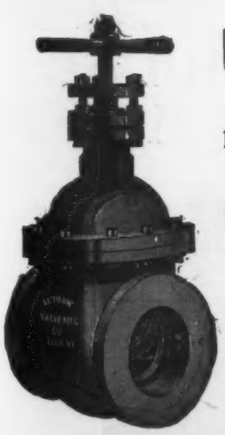

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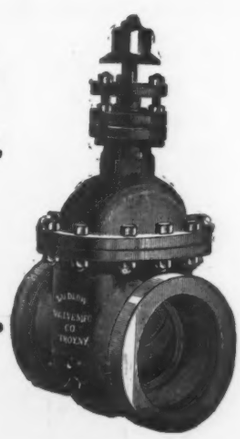

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HALIFAX, ENGLAND.

GAS ENGINEERS
AND CONTRACTORS.

PUBLIC LIGHTING TABLES FOR NOVEMBER, 1910.

[Communicated by The American Meter Company.]

Day of Week.	Table No. 1. FOLLOWING THE MOON. Date.	Light	Extinguish.	Day of Week.	Table No. 2. NEW YORK CITY. ALL NIGHT LIGHTING. Date.	Complete Lighting in 1 Hour.	Complete Extinguishing in 50 Minutes.
						From Time Given.	
		P.M.				P.M.	A.M.
Tue.	1	5.30 N.M.	5.30 A.M.	Tue.	1	4.37	5.17
Wed.	2	5.30	5.30	Wed.	2	4.37	5.17
Thu.	3	5.30	5.30	Thu.	3	4.37	5.17
Fri.	4	5.30	5.30	Fri.	4	4.37	5 17
Sat.	5	5.30	5.30	Sat.	5	4.27	5.27
Sun.	6	6.50	5.40	Sun.	6	4.27	5.27
Mon.	7	7.40	5.40	Mon.	7	4.27	5.27
Tue.	8	8.50	5.40	Tue.	8	4.27	5.27
Wed.	9	9.50 F.Q.	5.40	Wed.	9	4.27	5.27
Thu.	10	11.00	5.40	Thu.	10	4.27	5.27
		A.M.					
Fri.	11	12.10	5.40	Fri.	11	4.27	5.27
Sat.	12	1.20	5.40	Sat.	12	4.17	5.32
Sun.	13	2.30	5.40	Sun.	13	4.17	5.32
Mon.	14	3.50	5.40	Mon.	14	4.17	5.32
Tue.	15	No L.	No L.	Tue.	15	4.17	5.32
Wed.	16	No L. F.M.	No L.	Wed.	16	4.17	5.32
Thu.	17	No L.	No L.	Thu.	17	4.17	5.32
Fri.	18	5.10 P.M.	7.00 P.M.	Fri.	18	4.17	5.32
Sat.	19	5.10	8.00	Sat.	19	4.12	5.42
Sun.	20	5.10	9.10	Sun.	20	4.12	5.42
Mon.	21	5.10	10.20	Mon.	21	4.12	5.42
Tue.	22	5.10	11.30	Tue.	22	4 12	5.42
Wed.	23	5.10 L.Q.	12.40 A.M.	Wed.	23	4.12	5.42
Thu.	24	5.10	1.50	Thu.	24	4.12	5.42
Fri.	25	5.10	2.50	Fri.	25	4.12	5.42
Sat.	26	5.10	3.50	Sat.	26	4.07	5.52
Sun.	27	5.10	4.50	Sun.	27	4.07	5.52
Mon.	28	5.10	6.00	Mon.	28	4.07	5.52
Tue.	29	5.10	6.00	Tue.	29	4.07	5.52
Wed.	30	5.10	6.00	Wed.	30	4.07	5.52

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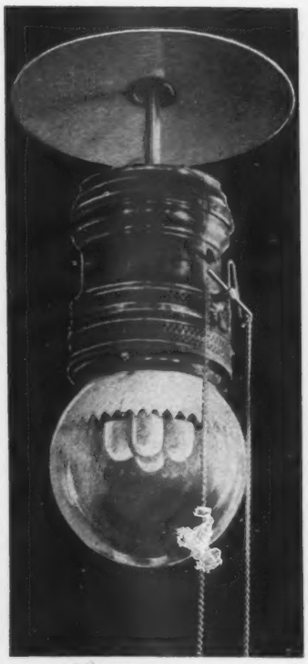

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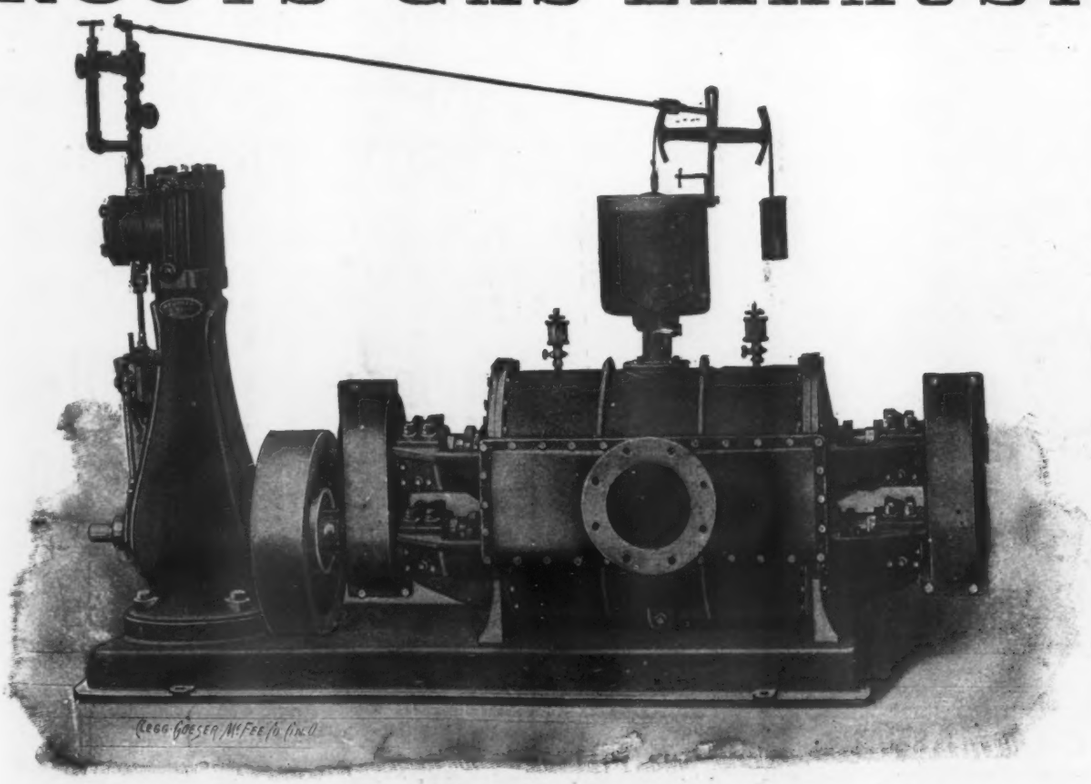

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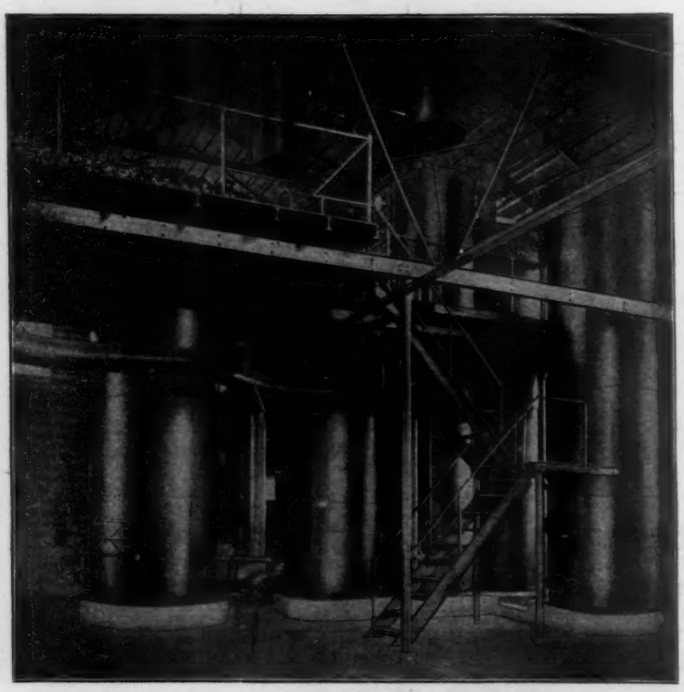

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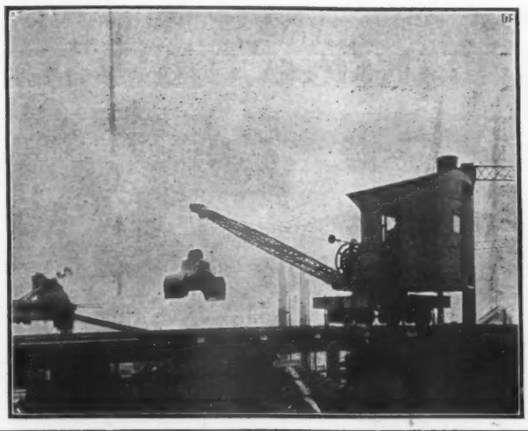

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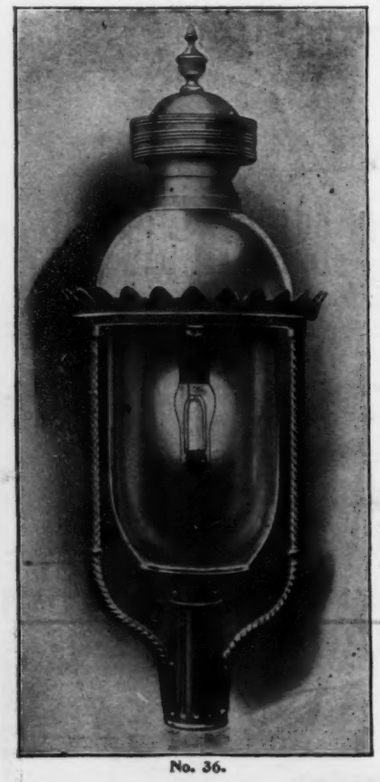

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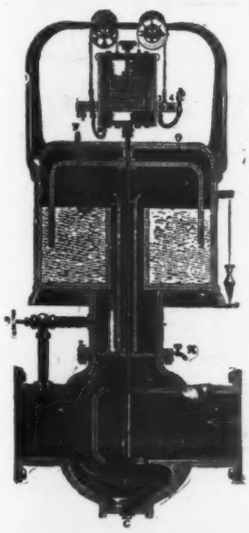

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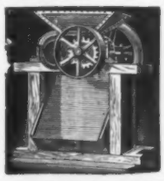

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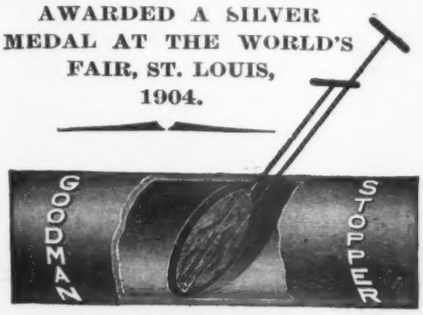

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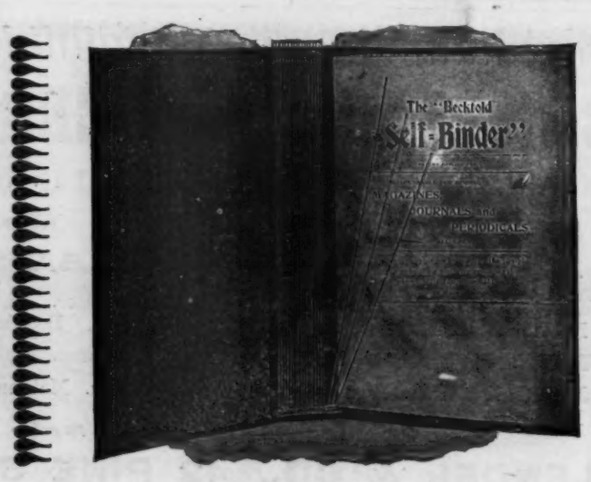

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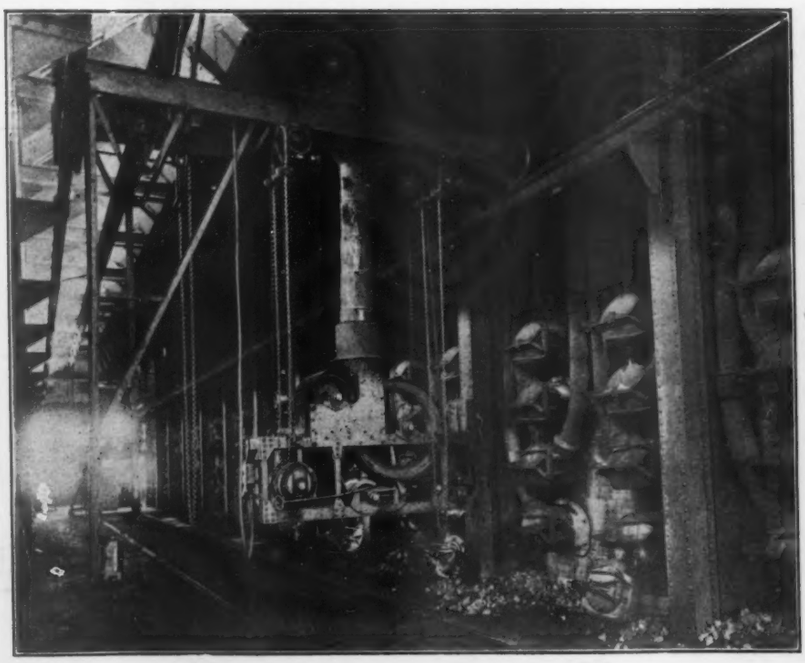

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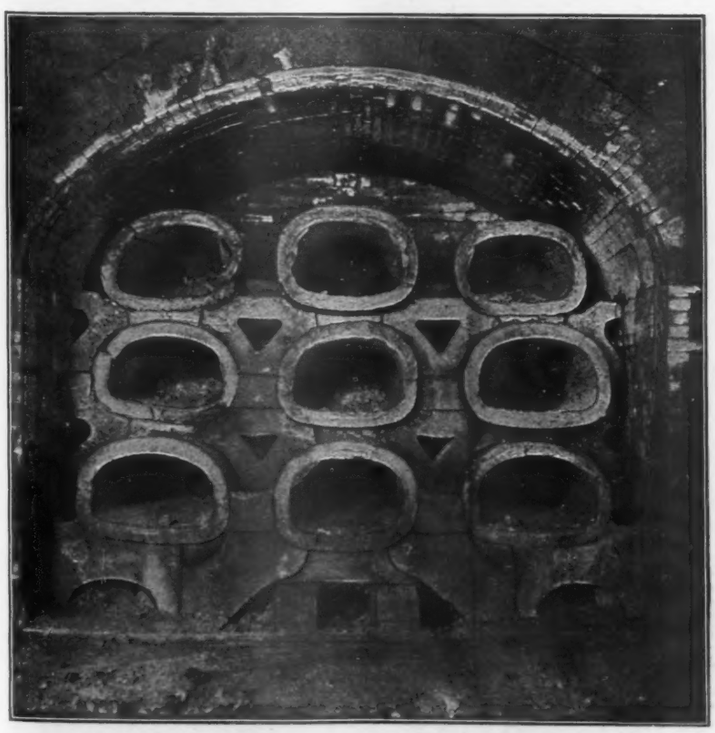

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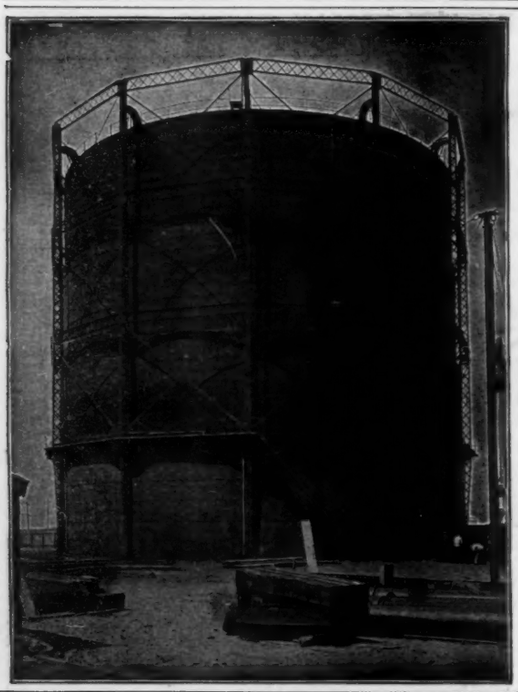

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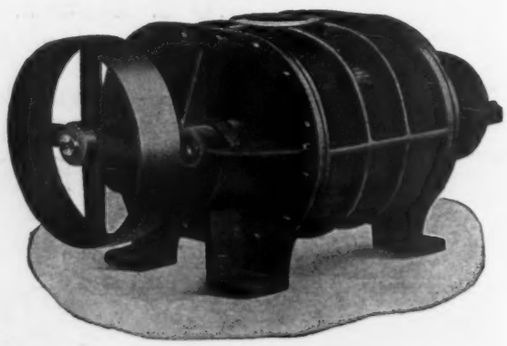

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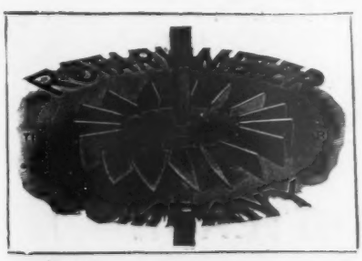

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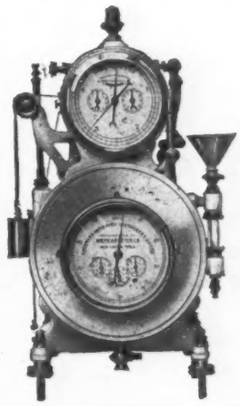

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